

Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### Vertical Interior Post and Horizontal Rail Design

No1 Date _	10/7/21		
	DDINIT	$H_{post} := 42 in$	Plans
CHECK I	KIINI	$F_y := 50 \text{ ksi}$	ASTM F1043
Dwg. Checked Aga		$E_s := 29000 \text{ ksi}$	
And Calc. Check C	Confirmed.	$F_{ii} = 58 \text{ ksi}$	
	10/7/01		
By	Date $\frac{10/7/21}{1}$	Pipe for Interior Posts	
	Date <u>12/22/2</u>		
	Date		
	Date	$t_{post} \coloneqq 0.160 \; in$	
Verified	Date		
Properties for ASTM F	·1043 IC 1-5/8" F	Pipe for Rails	
Rail OD:		$OD_{rail} := 1.66$ in	
Rail Thickness:		$t_{rail} := 0.111$ in	

Design Point Live Load  $P_{LL} := 200 \ lbf$ AASHTO 13.8.2 Design Uniform Live Load  $w_{IJ} := 50 \ plf$ AASHTO 13.8.2  $L_{spc} := 6 \, ft$ Plans  $f_{clf} := 0.48 \ psf$ 

Post spacing: Weight of chain link fence: Design wind load from chain link fence:  $f_{wind} := 15 \ psf$ 

PL Load Factor:	$\gamma_{PL} := 1.75$
DC Load Factor:	$\gamma_{DL} := 1.25$
WS Load Factor:	$\gamma_{WS} := 1.00$
Resistance Factors:	

Load Factors (AASHTO Tbl. 3.4.1-1):

Steel Flexure (AASHTO 6.5.4.2):  $\phi_f := 1.00$ Steel Shear (AASHTO 6.5.4.2):  $\phi_{v} := 1.00$ Tension, Yielding in Gross Section:  $\phi_v := 0.95$ Bending (AISC F1):  $\phi_b := 0.90$ Shear (AISC G1):  $\phi_{v\_AISC} := 0.90$ Bearing (AISC DG#1):  $\phi_{brg} := 0.60$ Fillet Weld (AISC Tbl. J2.5):  $\phi_{fw} := 0.75$ Bolts (AISC J3.6 & J3.7):  $\phi_{ab} := 0.75$ Adhesive Anchor Bolts (ACI 17.3.3,  $\phi_{adh} := 0.65$ Condition B, Category 1):

Steel weight density:  $\gamma_{steel} := 490 \ pcf$ 

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe															
Trade	Decima	I O.D.	Pipe \	wall			Sect	tion	Min. Y	Min. Yield		Max Bending	Calculated Load (lbs)		
Reference	Equiva	alent	Thickr	ess	Weig	ht	Modu	ulus	x Stren	gth	=	Moment	10' Free	Canti	lever
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches <sup>3</sup>	(mm³)	x psi	(Mpa)	=	Lb.ln.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x 50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x 50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x 50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x 50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x 50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x 50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X 50000	345	I	114295	3810	5486	1587
											П				

AASHTO 13.8.2

Output:

Post Section Properties:
Post inside diameter:
Post Area:

Post Unit Weight: Post centroid: Post Moment of Inertial:

Post Section Modulus:

Post Plastic Section Modulus:

Rail Section Properties: Rail inside diameter: Rail Area: Rail Unit Weight: Rail centroid: Rail Moment of Inertial: Rail Section Modulus:

Rail Plastic Section Modulus:

 $ID_{post} := OD_{post} - 2 \cdot t_{post}$  $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$ 

 $W_{post} := \gamma_{steel} \cdot A_{post}$  $c_{post} := 0.5 \cdot OD_{post}$ 

 $I_{post} := 0.049087 \cdot \left(OD_{post}^{4} - ID_{post}^{4}\right)$ 

 $Z_{post} := \frac{OD_{post}^{3} - ID_{post}^{3}}{6}$ 

 $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$  $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$  $W_{rail} := \gamma_{steel} \cdot A_{rail}$  $c_{rail} := 0.5 \cdot OD_{rail}$  $I_{rail} := 0.049087 \cdot \left( OD_{rail}^{4} - ID_{rail}^{4} \right)$ 

 $Z_{rail} := \frac{OD_{rail}^{3} - ID_{rail}^{3}}{6}$ 

 $ID_{post} = 2.555 in$ 

 $A_{post} = 1.365 \text{ in}^2$  $w_{post} = 4.644 \, plf$ 

 $c_{post} = 1.438 \ in$ 

 $I_{post} = 1.262 \ in^4$ 

 $S_{post} = 0.878 \text{ in}^3$ 

 $Z_{nost} = 1.181 \text{ in}^3$ 

 $ID_{rail} = 1.438 in$ 

 $A_{rail} = 0.54 \ in^2$ 

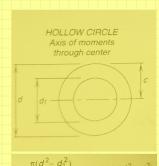
 $w_{rail} = 1.838 \, plf$  $c_{rail} = 0.83 in$ 

 $I_{rail} = 0.163 \text{ in}^4$ 

 $S_{rail} = 0.196 \text{ in}^3$ 

 $Z_{rail} = 0.267 \text{ in}^3$ 

 $\pi(d^4 - d_1^4)$ 





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 $\phi M_n = 59.038 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

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Post concentrated live load applied at top rail:	$P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.5 \text{ kip}$	$P_{post\_LL} = 0.5 \text{ kip}$	AASHTO Eqn. 13.8.2-1					
Post moment loading from live load:	$M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 21000 \ \textit{lbf} \cdot \textit{in}$	$M_{post\_LL} = 21000 \; lbf \cdot in$	Post treated as cantilevered beam					
Post shear from live load:	$V_{post\_LL} \coloneqq P_{post\_LL}$	$V_{post\_LL} = 0.5 \text{ kip}$	cantilevered beam					
Rail moment from live load applied:	$M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $f := \frac{H_{post} \cdot L_{spc}^{2}}{4}$		Rail treated as simply supported beam with vertical and horizontal live loads combined into					
Rail moment from dead load:	$M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{2}}{8}$	$M_{rail\_DL} = 144.615 \ lbf \cdot in$	resultant direction.					
Rail shear from live load:	$V_{rail\_LL} \coloneqq \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$	$V_{rail\_LL} = 0.312 \text{ kip}$						
Rail shear from dead load:	$V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{ctf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$	$V_{rail\_DL} = 0.008 \text{ kip}$	AASHTO load factors					
Factored Shear Load on Post:	$V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$	$V_{post\_u} = 0.875 \text{ kip}$	used instead of ASCE load factors found in AISC and ACI. This is					
Factored Moment Load on Post:	$M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$	$M_{post\_u} = 36750 \; lbf \cdot in$	acceptable as it is more conservative.					
Factored Shear Load on Rail:	$V_{rail\_u} \coloneqq \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$	$V_{rail\_u} = 0.556 \text{ kip}$	Vertical dead load was combined directly with live load resultant since					
Factored Moment Load on Rail:	$M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$	$M_{rail\_u} = 13162.928 \ lbf \cdot in$	it was so small compared to the live load.					
Post Analysis: Following AASHTO 6.12.1.2.3c for Shear Design: Gross Area:	$A_g \coloneqq A_{post}$	$A_g = 1.365 \ in^2$						
Distance from Max to 0 Shear:	$L_{v} \coloneqq H_{post}$	$L_{v} = 42  in$						
Critical Strength for Shear: $F_{cr} := min \left(0.58 \cdot F_y : \frac{1}{2} + \frac{1}{2$	$\max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}}}, \frac{0.78 \cdot E_s}{\sqrt{\frac{3}{OD_{post}}}} \right)^{\frac{5}{4}}, \frac{\frac{3}{OD_{post}}}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{2}{2}}} \right)$	F <sub>cr</sub> = 29 <b>ksi</b> AASHT & 6.12.1	O Eqns. 6.12.1.2.3c-2 1.2.3c-3					
Factored nominal shear resistance:	$\phi V_n \coloneqq \phi_v \cdot 0.5 \ F_{cr} \cdot A_g$	$\phi V_n = 19.788 \text{ kip} \qquad \text{AASH}$	ITO Eqn. 6.12.1.2.3c-1					
Post Shear Check:	$\frac{\phi V_n}{V_{post\_u}} = 22.615 \qquad Post\_Shear\_C$	$heck := if \frac{\phi V_n}{V_{post\_u}} \ge 1.0$   "Post shear strength else   "Post is not satisfact						
	Post_Shear_Ci	heck = "Post shear strength is sa	atisfactory."					
Following AASHTO 6.12.2.2.3 for Flexure Design:								
Check of Noncompact Section: Check_Compa	$act := if \frac{OD_{post}}{t_{post}} \le \frac{0.07 \cdot E_s}{F_y}$	D/t does not plastic modu 6.12.2.2.3-1	0 6.12.2.2.3, as long exceed 0.07E/Fy, lus and equation may be used.					
	"Section is not compact. Check wall slo	enderness."						
Check_Compo	act = "Section is compact. Local buckling does r	not apply."						

 $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ 

Factored Nominal Moment Resistance:



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Post\_Flex\_Check := if  $\frac{\phi M_n}{M_{post_u}} \ge 1.0$ "Post flexural strength is satisfactory." Post Flexural Check: "Post is not satisfactory."

Post Flex Check = "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area:

 $A_g = 0.54 \text{ in}^3$ 

Distance from Max to 0 Shear:

$$L_v := \frac{L_{spc}}{2}$$

Critical Strength for Shear:

$$F_{cr} \coloneqq \min \left( 0.58 \cdot F_y, \max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}} \left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{3}{2}}} \right) \right)$$

AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance:

$$\varphi_{r_n} := \varphi_{v} \circ 0.5 T_{c_n}$$

 $\phi V_n = 7.832 \text{ kip}$  AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check:

$$\frac{\phi V_n}{V_{rail\ u}} = 14.08$$

$$\frac{\phi V_n}{V_{rail\_u}} = 14.08 \qquad \qquad Rail\_Shear\_Check \coloneqq \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0$$
 
$$\parallel \text{``Rail shear strength is satisfactory.''}$$
 
$$\text{else}$$
 
$$\parallel \text{``Rail is not satisfactory.''}$$

Rail Shear Check = "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

Check\_Compact = "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance:

$$\phi M_n := \phi_f \cdot F_v \cdot Z_{rail}$$

$$\phi M_n = 13.339 \text{ kip} \cdot \text{in}$$
 AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{rail\ u}} = 1.013$$

$$\begin{aligned} \textit{Rail\_Flex\_Check} \coloneqq & \text{if } \frac{\phi M_n}{M_{rail\_u}} \geq 1.0 \\ & \quad \parallel \text{``Rail flexural strength is satisfactory.''} \\ & \quad \text{else} \\ & \quad \parallel \text{``Rail is not satisfactory.''} \end{aligned}$$

Rail Flex Check = "Rail flexural strength is satisfactory."



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Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post:

$$w_{post\ wind} := f_{wind} \cdot L_{spc}$$

$$w_{post\ wind} = 90\ plf$$

Design moment from wind on post:

$$M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$$

$$M_{post\_wind\_u} = 6615 \ lbf \cdot in$$

 $M_{post\ u} = 36750\ \textit{lbf} \cdot \textit{in} < - \text{LL controls}$ 

 $V_{post\ u} = 0.875\ \textit{kip}$  <- LL controls

$$V_{post\_wind\_u} := \gamma_{WS} \bullet w_{post\_wind} \bullet H_{post}$$

$$V_{post\_wind\_u} = 0.315$$
 kip

$$w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$$

$$w_{rail\ wind} = 26.25\ plf$$

$$M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$$

$$M_{rail\ wind\ u} = 1417.5\ lbf \cdot in$$

Design shear from wind on rail: 
$$V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$$

$$M_{rail\_u} = 13162.928 \, lbf \cdot in < - LL \text{ controls}$$

$$V_{rail\_wind\_u} = 0.079 \text{ kip}$$

$$V_{rail\ u} = 0.556\ \textit{kip}$$
 <- LL controls



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#### Base Plate Design - Line Post w/ Axial Compression

Given: Plans Cap width:

 $W_{cap} := 15.63 \ in$ Distance from post to end of cap:  $L_{end} := 72$  in

Plate thickness:  $t_n := 0.5 \ in$ 

Plate length (perpendicular to fence):  $N_{plate} := 8 in$ 

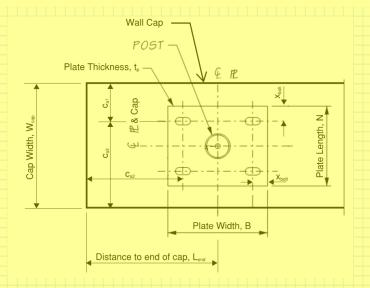
Plate width (parallel to fence):  $B_{nlate} := 10$  in

Compressive Strength of Concrete:  $f_c := 4 \text{ ksi}$ 

Side clearance to anchor bolts:  $x_{bolt} := 1.5 in$ 

Base plate steel yield strength:  $F_{v plate} := 36$  ksi

Number of rails:  $n_{rail} := 2$ 



Output:

Plate Area:  $A_{plate} = 80 \text{ in}^2$  $A_{plate} := N_{plate} \cdot B_{plate}$ 

 $c_{al} := \frac{1}{2} \left( W_{cap} - N_{plate} \right) + x_{bolt}$  $c_{al} = 5.315$  in Distance from bolt to near face of cap:  $c_{a2} \coloneqq L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ 

 $c_{a2} = 68.5$  in Distance from outside bolt to end of cap:

Distance from bolt to far face of cap:  $c_{a3} \coloneqq W_{cap} - c_{a1}$  $c_{a3} = 10.315$  in

Bearing Area taken to Be Same as Plate Area:  $A_{bearing} := A_{plate}$  $A_{bearing} = 80 \text{ in}^2$ bearing area to the same as the plate.

 $f_{pu\_max} := \phi_{brg} \cdot min \left[ 0.85 \cdot f_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f_c \right]$  $f_{pu\ max} = 2.04 \ ksi$ Max allowed bearing pressure: ACI Tbl. 14.5.6.1

 $q_{max} = (2.448 \cdot 10^5) \frac{lbf}{ft}$ Max allowed bearing pressure line:  $q_{max} := f_{pu\_max} \cdot B_{plate}$ 

Post dead load on plate  $P_{post\ DL} := w_{post} \cdot H_{post}$  $P_{post DL} = 0.016 \ kip$ 

Rail dead load on plat:  $P_{rail\ DL} := n_{rail} \cdot 2 \cdot V_{rail\ DL}$  $P_{rail\ DL} = 0.032 \ kip$ 

 $P_u := \gamma_{DL} \cdot (P_{nost DL} + P_{rail DL})$ Factored vertical load on plate:  $P_{y} = 0.06 \ kip$ 

 $Y_{min} := \frac{P_u}{a_{\cdots}}$ Minimum length of area of bearing:  $Y_{min} = 0.003$  in

 $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ Critical eccentricity distance:  $e_{crit} = 3.999 in$ AISC DG#1 Eqn. 3.3.7

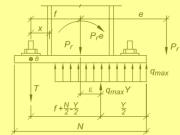
 $e_{loading} := \frac{M_{post\_u}}{P}$ Eccentricity of loading:  $e_{loading} = 607.565$  in AISC DG#1 Eqn. 3.3.6

Small moment check:

Small Moment\_Check := if  $e_{loading} \le e_{crit}$ "Moment is small, no need for anchor bolts." "Moment is large, need anchor bolts."

Small Moment Check = "Moment is large, need anchor bolts."

 $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$   $f_{dim} = 2.5$  in Distance from bolt to center of post:



Conservatively setting

AISC DG#1 Eqn. 3.3.3

Fig. 3.4.1. Base plate with large moment.



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Plate dimension check

$$\begin{aligned} \textit{Plate\_Dim\_Check} \coloneqq & \text{if } \left( f_{\textit{dim}} + \frac{N_{\textit{plate}}}{2} \right)^2 \geq \frac{2 \cdot P_u \cdot \left( e_{\textit{loading}} + f_{\textit{dim}} \right)}{q_{\textit{max}}} \\ & \text{ "Plate dimensions are OK."} \\ & \text{else} \\ & \text{ "Plate needs to be longer and/or wider."} \end{aligned}$$

Plate Dim Check = "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} \coloneqq \left( f_{dim} + \frac{N_{plate}}{2} \right) - \sqrt{\left( f_{dim} + \frac{N_{plate}}{2} \right)^2 - \frac{2 \cdot P_u \cdot \left( e_{loading} + f_{dim} \right)}{q_{max}}} \qquad Y_{loading} = 0.285 \text{ in}$$

$$AISC DG#1 Eqn. 3.4.3$$

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$$T_u = 5.744 \ kip$$

 $T_{y} = 5.744 \text{ kip}$  AISC DG#1 Eqn. 3.4.2

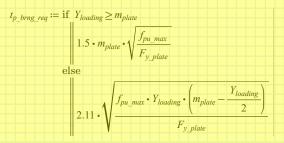
Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} \coloneqq \frac{N_{plate} - 0.8 \ OD_{post}}{2}$$

$$m_{plate} = 2.85 \ in$$

Calculating minimum thickness based on bearing:



 $t_{p \ brng \ req} = 0.441 \ in$ 

AISC DG#1 Egns. 3.3.14a-2 &

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$$x_{ten} = 1.35 in$$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}} \qquad t_{p\_ten\_req} = 0.31 \text{ in}$$

$$n_{p, ten, rea} = 0.31$$
 in AISC DG#1 Eqn. 3.4.7a

Controlling minimum required base plate thickness:  $t_{p \ req} := \max (t_{p \ brng \ req}, t_{p \ ten \ req})$ 

$$t_{p\_req} := \max \left( t_{p\_brng\_req}, t_{p\_ten\_req} \right)$$

$$t_{p \ req} = 0.441 \ in$$

Check chosen plate thickness:

$$Plate\_Thick\_Check \coloneqq \text{if } t_p \ge t_{p\_req}$$
 \( \big| "Chosen plate thickness is adequate." \\ else \\ \big| "Need a thicker plate."

Plate Thick Check = "Chosen plate thickness is adequate."



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#### Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size:

Min fillet weld size based on AISC Table J2-4

Chosen fillet weld size

Weld material:

 $F_{EXX} := 70 \text{ ksi}$ 

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter:

Effective Length of Weld:

Effective Throat Thickness:

Area of Weld:

Moment of Inertia of circular fillet weld:

Polar moment of Inertia of circular fillet weld:

 $L_g := \pi \cdot OD_{post}$ 

 $A_w := L_w \cdot t_e$ 

 $L_g = 9.032$  in

 $L_w = 8.407 in$ 

 $t_e := min\left(w \cdot \sin\left(45 \ deg\right), \frac{L_w}{4}\right)$   $t_e = 0.221 \ in$ 

 $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad I_w = 2.062 \text{ in}^4$ 

 $J_w := 2 \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad J_w = 4.124 \text{ in}^4$ 

AISC, Sect. J2, Pts. 2a & 2b

AISC, Sect. J2, Pts. 2a

17. Very thin annulus

 $A = 2\pi Rt$  $I_{\nu} = I_{\nu} = \pi R^3 t$  $r_{y} = r_{y} = 0.707R$  $J = 2\pi r^3 t$ 

Determine design strength of weld:

Nominal strength of weld metal:

 $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ 

 $F_w = 31.5 \ ksi$ 

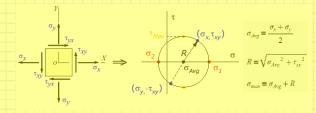
AISC, Tbl. J2.5

Normal stress caused by bending moment:

 $\sigma_b = 25.619 \text{ ksi}$   $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$ 

Stress caused by shearing force:

Resultant stress in weld from loading:



Check of weld thickness:

Weld\_Design\_Check := if  $F_w \ge \sigma_{max}$ 

"Chosen weld size is sufficient."

"Need bigger fillet weld."

Weld Design Check = "Chosen weld size is sufficient."



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#### **Anchor Bolt Connection Design**

Given:

Number of anchor bolts resisting loads:

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter:

Bolt area:

Bolt nominal yield stress strength:

Bolt nominal ultimate tensile stress strength:

Bolt embedment:

Output:

Tension anchor bolt spacing:

Bolt nominal tensile stress strength:

Bolt nominal shear stress strength:

Ultimate tension load on one anchor bolt:

Required shear stress on one bolt:

Bolt modified nominal tensile stress strength, modified for effects of shear stress:

Bolt factored tensile resistance:

Check of bolt tensile stress:

Only one side's bolts resist tension or shear.

 $d_{ab} \coloneqq \frac{5}{8} in$ 

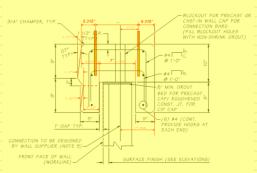
Plans

 $A_b := 0.307 \ in^2$ AISC Tbl. 7-18

 $F_{v bolt} := 36$  ksi

AISC Tbl. 2-3

 $F_{u \ bolt} := 58 \ ksi$  $h_{ef} := 5$  in



WALL CAP TYPICAL DETAIL

 $s_I := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1} \qquad s_I = 7 \text{ in}$ 

 $F_{nv} := 0.40 \cdot F_{u \ bolt}$ 

AISC Tbl. J3.2

AISC Tbl. J3.2, assuming threads within shear plane

 $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$   $F_{nt}' = 43.5 \text{ ksi}$  AISC Eqn. J3-3a

 $\phi R_{n \ bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$   $\phi R_{n \ bolt} = 10.016 \ kip$ 

AISC Eqn. J3-2

ACI 17.2.1.1

*Bolt\_Tensile\_Check* = "Bolt is satisfactory."

 $Bolt\_Tensile\_Check := if \phi R_{n \ bolt} \ge T_{u \ ab}$ 

"Bolt is satisfactory."

"Bolt is no good."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor:

 $d_a := d_{ab}$ 

 $d_a = 0.625$  in

Critical edge distance for adhesive anchors:

 $c_{ac} \coloneqq 2 h_{ef}$   $c_{ac} = 10 in$ 

ACI 17.7.6

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout:

*Group Tension Breakout Check* := if  $s_1 \le 3 \cdot h_{ef}$ 

"Bolts act in group."

"Bolts act singly."

Group Tension Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

 $A_{Nco} := 9 \cdot h_{ef}^2$ 

 $A_{Nco} = 225 \ in^2$ 

ACI Eqn. 17.4.2.1c

Actual projected influence area for bolt(s):  $A_{Nc} := min\left(\left(c_{al} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_{1}, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$   $A_{Nc} = 281.93 \ \text{in}^{2} \quad A_{Cl} = 281.93 \ \text{in}^{2} \quad$ 

Concrete tension breakout strength coefficient:

Value of 17 for post-installed anchors, per ACI 17.4.2.2



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Basic concrete tension breakout strength for single anchor:

$$N_b \coloneqq k_c \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$$
  $N_b = 12.021 \ kip$  ACI Eqn. 17.4.2.2a

Factor for eccentrically loaded anchor bolts:

 $\Psi_{ec\ N} := 1.0$ 

Anchor bolts are not loaded eccentrically.

ACI 17.4.2.4

Factor for anchor bolts near an edge:

$$\varPsi_{ed\_N} \coloneqq min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{1.5 \cdot h_{ef}}\right)$$

 $\Psi_{ed\ N} = 0.913$  ACI Eqn. 17.4.2.5b

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{c,N} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.4.2.6

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_N} := min\left(1.0 \text{ , max}\left(\frac{c_{al}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right) \qquad \Psi_{cp\_N} = 0.75 \qquad \text{ACI Eqn. 17.4.2.7b}$$

Nominal concrete tension breakout strength:

$$\phi N_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b \qquad \phi N_{cbg} = 9.382 \text{ kip} \qquad \text{ACI Eqn. 17.4.2.1b}$$

Check of concrete tension breakout failure:

Concrete\_Tension\_Breakout\_Check := if 
$$\phi N_{cbg} \ge n_{ab} \cdot T_{u\_ab}$$

"Bolt is satisfactory."

else

"Bolt is no good."

Concrete Tension Breakout Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500 \ psi}\right)^{0.1} \cdot 2220 \ psi = 1512.441 \ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500 \text{ psi}}\right)^{0.15} \cdot 2210 \text{ psi} = 1541.429 \text{ psi}$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below

Minimum bond stress strength:

$$\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right) \qquad \qquad \tau_{uncr} = 1512.441 \text{ psi}$$

$$\tau_{max} = 1512.441 \ psi$$

Distance to edge of project influence area:

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100 \text{ nsi}}}$$

 $c_{Na} = 7.329$  in

ACI Egn. 17.4.5.1d

Check if anchor bolts act in group for bond failure:

$$Group\_Bond\_Failure\_Check := if s_1 \le 2 c_{Na}$$

Group Bond Failure Check = "Bolts act in group."



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Theoretical projected influence area of a single bolt far from an edge:

$$A_{Nao} := (2 c_{Na})$$

 $A_{Nao} = 214.835 \text{ in}^2$ 

ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s):

$$A_{Na} := min\left(\left(c_{Na} + min\left(s_{1}, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$$

 $A_{Na} = 273.826 \text{ in}^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor:

$$N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$$

 $N_{ba} = 14.848 \ kip$ 

ACI Ean. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec\_Na} := 1.0$$

 $\Psi_{ec,Na} := 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.4.5.3

Factor for anchor bolts near an edge:

$$\Psi_{ed\_Na} := min \left( 1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{c_{Na}} \right)$$

$$\Psi_{ed\_Na} = 0.918$$

ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_Na} := min\left(1.0 \text{, max}\left(\frac{c_{al}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$$

 $\Psi_{cp\_Na}=0.733$  ACI Eqn. 17.4.5.5b Wall caps are not under load, and per the wall cap design, vice moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s):

$$\phi N_{ag} \coloneqq \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba} \qquad \phi N_{ag} = 8.272 \text{ kip}$$

Check of bolt bond stress failure:

$$Bond\_Stress\_Check \coloneqq \text{if } \phi N_{ag} \geq n_{ab} \cdot T_{u\_ab}$$
 \text{"Bolt is satisfactory."} else \text{"Bolt is no good."}

Bond Stress Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout:

$$\begin{aligned} \textit{Group\_Shear\_Breakout\_Check} \coloneqq & \text{if } s_l \leq 3 \cdot c_{al} \\ & & \| \text{``Bolts act in group.''} \end{aligned}$$

ACI 17.2.1.1

Group Shear Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Vco} \coloneqq 4.5 \cdot c_{al}^{2}$$

$$A_{Vco} = 127.122 \text{ in}^2$$

"Bolts act singly."

ACI Eqn. 17.5.2.1c

Actual projected influence area for bolt(s): 
$$A_{Vc} := min\left(1.5 \cdot c_{al} \cdot \left(1.5 \cdot c_{al} + min\left(s_1, 3 \cdot c_{al}\right) + min\left(1.5 \cdot c_{al}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$$
  $A_{Vc} = 182.929 \ in^2$  ACI Fig. R17.5.2.1b

Load bearing length:

$$l_e := h_{ef}$$

$$l_e = 5$$
 in

ACI 17.5.2.2

Basic concrete breakout strength in shear for single anchor:

$$V_b \coloneqq \min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5 \ \textit{kip}$$

Concrete is not light weight;

ACI Eqns. 17.5.2.2a

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec}$$
  $_{V} := 1.0$ 

 $\Psi_{ec\ V} \coloneqq 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.5.2.5

Factor for anchor bolts near an edge:

$$\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$$

$$\Psi_{ed_{V}} = 1$$

ACI Eqns. 17.5.2.6a

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{cV} := 1.4$ 

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.5.2.7



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Factor for small embedment

$$\varPsi_{h\_V} \coloneqq \min\left(1.0 \; , \sqrt{\frac{1.5 \cdot c_{al}}{h_{ef}}}\right) \qquad \varPsi_{h\_V} = 1 \qquad \qquad \text{ACI Eqn. 17.5.2.8}$$

Nominal concrete shear breakout strength:

$$\phi V_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec} \cdot \Psi_{ed} \cdot \Psi_{c} \cdot \Psi_{h} \cdot \Psi_{b} \qquad \phi V_{cbg} = 8.512 \text{ kip} \qquad \text{ACI Eqn. 17.5.2.1b}$$

$$Concepts Shage Procleant Check = 14 \text{ AV} > V$$

Check of concrete shear breakout failure:

Concrete\_Shear\_Breakout\_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear:

$$\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$$

 $\phi N_{cpg} = 8.272 \text{ kip}$ 

ACI 17.5.3.1

ACI 17.5.3.1

Concrete pryout strength in shear coefficient:

Nominal concrete pryout strength of anchor(s) in shear:

$$\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$$

 $\phi V_{cpg} = 16.545 \ kip$ 

ACI Eqn. 17.5.3.1b

Check of concrete pryout strength in shear:

$$\begin{aligned} \textit{Concrete\_Shear\_Pryout\_Check} \coloneqq & \text{if } \phi \textit{V}_\textit{cpg} \geq \textit{V}_\textit{post\_u} \\ & \text{``Bolt is satisfactory.''} \\ & \text{else} \\ & \text{``Bolt is no good.''} \end{aligned}$$

Concrete Shear Pryout Check = "Bolt is satisfactory."





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3.4.1-1):

Vertical Interior Post and Horizon	ital Rail Design		
Given:			Load Factors (AASHTO Tbl.
Post Height:	$H_{post} := 66$ in	Plans	PL Load Factor:
Step Height:	$H_{step} := 24$ in		DC Load Factor:
Post and Rail Yield Strength:	$F_y := 50 \text{ ksi}$	ASTM F1043	WS Load Factor:
Post and Rail Modulus of Elasticity:	$E_s := 29000 \ ksi$		
Post and Rail Ultimate Strength:	$F_u := 58 \text{ ksi}$		Resistance Factors:
			Steel Flexure (AASHTO 6.5.4

 $P_{IJ} := 200 \ lbf$ 

 $w_{IJ} := 50 \ plf$ 

Properties for ASTM F1043 IC 2-7/8" Pipe for Interior Posts Post OD:  $OD_{post} := 2.875 in$ Post Thickness:  $t_{post} := 0.160 \ in$ 

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails Rail OD:

 $OD_{rail} := 1.660 \ in$ Rail Thickness:  $t_{rail} := 0.111 \ in$ 

Design Point Live Load Design Uniform Live Load Post spacing:

 $L_{spc} := 4 \, ft$ Weight of chain link fence:  $f_{clf} := 0.48 \ psf$ Design wind load from chain link fence:  $f_{wind} := 15 \, psf$ 

 $\gamma_{PL} := 1.75$  $\gamma_{DL} := 1.25$  $\gamma_{WS} := 1.00$ .4.2):  $\phi_f := 1.00$ Steel Shear (AASHTO 6.5.4.2):  $\phi_{v} := 1.00$ Tension, Yielding in Gross Section:  $\phi_v := 0.95$ Bending (AISC F1):  $\phi_b := 0.90$ Shear (AISC G1):  $\phi_{v AISC} := 0.90$ Bearing (AISC DG#1):  $\phi_{brg} := 0.60$ Fillet Weld (AISC Tbl. J2.5):  $\phi_{fw} := 0.75$ Bolts (AISC J3.6 & J3.7):  $\phi_{ab} := 0.75$ Adhesive Anchor Bolts (ACI 17.3.3,  $\phi_{adh} := 0.65$ Condition B, Category 1):

Steel weight density:  $\gamma_{steel} := 490 \ pcf$ 

ASTM F1	ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe														
Trade	Decima	I O.D.	Pipe \	wall				ion	Min. Y	Min. Yield		Max Bending	Calculated Load (lbs)		
Reference	Equiva	alent	Thickr	ess	Weig	ht	Modu	ılus	x Stren	gth	=	Moment	10' Free	Canti	lever
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches <sup>3</sup>	(mm³)	x psi	(Mpa)	=	Lb.ln.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x 50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x 50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x 50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x 50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x 50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x 50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	50000	345		114295	3810	5486	1587
											П				

AASHTO 13.8.2

AASHTO 13.8.2

AASHTO 13.8.2

Output:

Post Section Properties:

Post inside diameter:  $ID_{post} := OD_{post} - 2 \cdot t_{post}$  $ID_{post} = 2.555 in$ Post Area:  $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$  $A_{post} = 1.365 \text{ in}^2$ Post Unit Weight:  $w_{post} = 4.644 \, plf$  $W_{post} := \gamma_{steel} \cdot A_{post}$  $c_{post} = 1.438 in$ Post centroid:  $c_{post} := 0.5 \cdot OD_{post}$  $I_{post} = 1.262 \text{ in}^4$ Post Moment of Inertial:  $I_{post} := 0.049087 \cdot \left(OD_{post}^{4} - ID_{post}^{4}\right)$ Post Section Modulus:  $S_{nost} = 0.878 \text{ in}^3$ 

 $Z_{post} := \frac{OD_{post}^{3} - ID_{post}^{3}}{6}$ Post Plastic Section Modulus:  $Z_{nost} = 1.181 \text{ in}^3$ 

Rail Section Properties:

Rail inside diameter:  $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$  $ID_{rail} = 1.438 in$ Rail Area:  $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$  $A_{rail} = 0.54 \ in^2$ Rail Unit Weight:  $W_{rail} := \gamma_{steel} \cdot A_{rail}$  $w_{rail} = 1.838 \, plf$  $c_{rail} = 0.83 in$ Rail centroid:  $c_{rail} := 0.5 \cdot OD_{rail}$ Rail Moment of Inertial:  $I_{rail} := 0.049087 \cdot \left(OD_{rail}^{4} - ID_{rail}^{4}\right)$  $I_{rail} = 0.163 \text{ in}^4$ Rail Section Modulus:  $S_{rail} = 0.196 \text{ in}^3$ 

 $Z_{rail} := \frac{OD_{rail}^{3} - ID_{rail}^{3}}{C}$ Rail Plastic Section Modulus:

HOLLOW CIRCLE

 $\pi(d^4 - d_1^4)$ 

 $Z_{rail} = 0.267 \text{ in}^3$ 



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Post concentrated live load applied at high top rail:	$P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.3 \text{ kip}$	$P_{post\_LL\_H} = 0.3 \text{ kip}$	AASHTO Eqn. 13.8.2-1, modified for split top rails
Post concentrated live load applied at low top rail:	$P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0.1 \text{ kip}$	$P_{post\_LL\_L} = 0.1 \text{ kip}$	
Post moment loading from live load: $M_{post\_LL} := P_{post\_LL}$	$_{ost\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot (H_{post} - H_{step})$	$M_{post\_LL} = 24000 \; lbf \cdot in$	Post treated as cantilevered beam
Post shear from live load:	$V_{post\_LL} \coloneqq P_{post\_LL\_H} + P_{post\_LL\_L}$	$V_{post\_LL} = 0.4 \text{ kip}$	
Rail moment from live load applied:	$M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^2}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$	$M_{rail\_LL} = 4097.056 \ lbf \cdot ii$	and horizontal live
Rail moment from dead load:	$M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{2}}{8}$	$M_{rail\_DL} = 75.793 \ lbf \cdot in$	loads combined into resultant direction.
Rail shear from live load:	$V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$	$V_{rail\_LL} = 0.241 \text{ kip}$	
Rail shear from dead load:	$V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$	$V_{rail\_DL} = 0.006 \text{ kip}$	
Nali Sileai Ilotti dead load.	$V_{rail\_DL} := {2} + {2}$	$V_{rail\_DL} = 0.000 \text{ kip}$	AASHTO load factors
Factored Shear Load on Post:	$V_{post\_u} \coloneqq \gamma_{PL} \cdot V_{post\_LL}$	$V_{post\_u} = 0.7 \text{ kip}$	used instead of ASCE load factors found in AISC and ACI. This is
Factored Moment Load on Post:	$M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$	$M_{post\_u} = 42000 \; lbf \cdot in$	acceptable as it is more conservative.
Factored Shear Load on Rail:	$V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$	$V_{rail\_u} = 0.43 \text{ kip}$	Vertical dead load was combined directly with live load resultant since
Factored Moment Load on Rail:	$M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$	$M_{rail\_u} = 7264.59 \ lbf \cdot in$	it was so small compared to the live load.
Post Analysis:			
Following AASHTO 6.12.1.2.3c for Shear Design:			
Gross Area:	$A_g := A_{post}$	$A_g = 1.365 \text{ in}^2$	
Distance from Max to 0 Shear:		$L_{\nu} = 66$ in	
Critical Strength for Shear: $F_{cr} := min \left(0.58 \cdot F_y, \text{m}\right)$	$ \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}} \left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right) $	F <sub>cr</sub> = 29 <b>ksi</b> AASH & 6.12	ITO Eqns. 6.12.1.2.3c-2 2.1.2.3c-3
Factored nominal shear resistance:	$\phi V_n \coloneqq \phi_v \cdot 0.5 \ F_{cr} \cdot A_g$	$\phi V_n = 19.788 \text{ kip} \qquad AAS$	SHTO Eqn. 6.12.1.2.3c-1
Post Shear Check:	$\frac{\phi V_n}{V_{post\ u}} = 28.269 \qquad Post\_Shear\_Ch$	$eck := if \frac{\phi V_n}{V_{post\_u}} \ge 1.0$ "Post shear strength of the strength	
	' post_u	"Post shear strengt	th is satisfactory."
		else "Post is not satisfa	
	Post_Shear_Ch	eck = "Post shear strength is	satisfactory."
Following AASHTO 6.12.2.2.3 for Flexure Design:			
Check of Noncompact Section: Check_Compact	$t := \text{if } \frac{OD_{post}}{t_{post}} \le \frac{0.07 \cdot E_s}{F_y}$		TO 6.12.2.2.3, as long of exceed 0.07E/Fy,
	$t_{post}$ $F_y$ "Section is compact. Local buckling do	plastic mod	dulus and equation 1 may be used.
	else		
	"Section is not compact. Check wall sle	nderness.	
Check_Compact	t = "Section is compact. Local buckling does n	ot apply."	



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Factored Nominal Moment Resistance:

$$\phi M_n := \phi_f \cdot F_v \cdot Z_{nost}$$

 $\phi M_n = 59.038 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{post\ u}} = 1.406$$

$$\frac{\phi M_n}{M_{post\_u}} = 1.406$$

$$Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \ge 1.0$$

$$\text{"Post flexural strength is satisfactory."}$$

$$\text{else}$$

$$\text{"Post is not satisfactory."}$$

Post\_Flex\_Check = "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area:

$$A_g := A_{rail}$$

$$A_g = 0.54 \text{ in}^2$$

Distance from Max to 0 Shear:

$$L_v := \frac{L_{spc}}{2}$$

$$L_v = 24$$
 in

Critical Strength for Shear:

$$F_{cr} \coloneqq \min \left( 0.58 \cdot F_y, \max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}} \left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{3}{2}}} \right) \right)$$

F<sub>cr</sub> = 29 **ksi** AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance:

$$\phi V_n := \phi_v \cdot 0.5 \ F_{cr} \cdot A_g$$

$$\phi V_{n} = 7.832 \ kip$$

 $\phi V_n = 7.832 \text{ kip}$  AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check:

$$\frac{\phi V_n}{V_{rail\_u}} = 18.199$$

$$\frac{\phi V_n}{V_{rail\_u}} = 18.199$$

$$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0$$

$$\text{"Rail shear strength is satisfactory."}$$

$$\text{else}$$

$$\text{"Rail is not satisfactory."}$$

Rail Shear Check = "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$$\begin{aligned} \textit{Check\_Compact} \coloneqq & \text{if } \frac{OD_{\textit{rail}}}{t_{\textit{rail}}} \leq \frac{0.07 \cdot E_s}{F_y} \\ & \parallel \text{"Section is compact. Local buckling does not apply."} \\ & \text{else} \\ & \parallel \text{"Section is not compact. Check wall slenderness."} \end{aligned}$$

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

Check Compact = "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance:

$$\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$$

 $\phi M_n = 13.339 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{rail\_u}} = 1.836$$

Rail Flex Check = "Rail flexural strength is satisfactory."



By: MJF Chk: RFA On: 9/17/2021 On: 12/22/2021

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post:

$$w_{post\_wind} := f_{wind} \cdot L_{spc}$$

$$w_{post\_wind} = 60 \, plf$$

Design moment from wind on post:

$$M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$$

$$M_{post\_wind\_u} = 10890 \ lbf \cdot in$$

$$V_{post\ wind\ u} := \gamma_{WS} \cdot w_{post\ wind} \cdot H_{post}$$

$$V_{post\ wind\ u} = 0.33$$
 kip

$$V_{post\ u} = 0.7\ kip$$
 <- LL controls

 $M_{post\ u} = 42000\ \textit{lbf} \cdot \textit{in} < - \ \text{LL controls}$ 

Uniform wind on rail:

$$w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$$

$$w_{rail\ wind} = 41.25\ plf$$

Design moment from wind on rail:

$$M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$$

$$M_{rail\_wind\_u} = 990$$
 *lbf* • *in*

$$V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$$

$$V_{rail\ wind\ u} = 0.083\ kip$$

$$V_{rail\ u} = 0.43\ kip$$
 <- LL controls

 $M_{rail\ u} = 7264.59\ lbf \cdot in <- LL controls$ 



Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### Base Plate Design - Line Post w/ Axial Compression

Given: Cap width: Plans

 $W_{cap} := 15.63 \ in$ 

Distance from post to end of cap:

 $L_{end} := 8$  in  $t_n := .5 \ in$ 

Plate length (perpendicular to fence):

 $N_{plate} := 8 in$ 

Plate width (parallel to fence):

 $B_{plate} := 10 in$ 

Compressive Strength of Concrete: Side clearance to anchor bolts:

 $f_c := 4 \, ksi$  $x_{bolt} := 1.5 \ in$ 

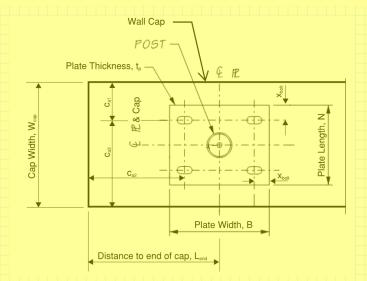
 $n_{rail} := 4$ 

Base plate steel yield strength:

 $F_{v plate} := 36$  ksi

Number of rails:

Plate thickness:



#### Output:

Plate Area:

 $A_{plate} := N_{plate} \cdot B_{plate}$ 

 $A_{plate} = 80 \text{ in}^2$ 

Distance from bolt to near face of cap:

 $c_{al} := \frac{1}{2} \left( W_{cap} - N_{plate} \right) + x_{bolt}$ 

 $c_{al} = 5.315$  in

Distance from outside bolt to end of cap:

Bearing Area taken to Be Same as Plate Area:

 $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ 

 $c_{a2} = 4.5$  in

Distance from bolt to far face of cap:

 $c_{a3} := W_{cap} - c_{a1}$  $A_{bearing} := A_{plate}$ 

 $c_{a3} = 10.315$  in  $A_{bearing} = 80 \text{ in}^2$ 

Conservatively setting bearing area to the same as the plate.

Max allowed bearing pressure:

Post dead load on plate

Rail dead load on plat:

$$f_{pu\_max} := \phi_{brg} \cdot min \left( 0.85 \cdot f_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}} \right., 1.7 \cdot f_c \right)$$

 $f_{pu max} = 2.04 \, ksi$ 

 $q_{max} = (2.448 \cdot 10^5) \frac{lbf}{ft}$ 

ACI Tbl. 14.5.6.1

Max allowed bearing pressure line:

 $q_{max} := f_{pu\_max} \cdot B_{plate}$ 

 $P_{post\ DL} := w_{post} \cdot H_{post}$ 

 $P_{rail\ DL} := n_{rail} \cdot 2 \cdot V_{rail\ DL}$ 

 $P_{rail\ DL} = 0.051 \ kip$ 

 $P_{post\ DL} = 0.026$  kip

Factored vertical load on plate:

 $P_u := \gamma_{DL} \cdot (P_{post\ DL} + P_{rail\ DL})$ 

 $P_{y} = 0.095 \text{ kip}$ 

Minimum length of area of bearing:

 $Y_{min} := \frac{P_u}{q}$ 

 $Y_{min} = 0.005 in$ AISC DG#1 Eqn. 3.3.3

Critical eccentricity distance:

 $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ 

 $e_{crit} = 3.998$  in

AISC DG#1 Eqn. 3.3.7

Eccentricity of loading:

 $e_{loading} := \frac{M_{post\_u}}{P}$ 

 $e_{loading} = 441.7$  in

AISC DG#1 Eqn. 3.3.6

Small moment check:

Small Moment Check := if  $e_{loading} \le e_{crit}$ 

"Moment is small, no need for anchor bolts."

"Moment is large, need anchor bolts."

Small Moment Check = "Moment is large, need anchor bolts."

Distance from bolt to center of post:

$$f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$$
  $f_{dim} = 2.5$  in

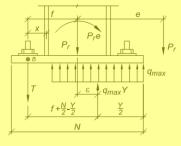


Fig. 3.4.1. Base plate with large moment.



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Plate dimension check

Plate\_Dim\_Check = "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} \coloneqq \left( f_{dim} + \frac{N_{plate}}{2} \right) - \sqrt{\left( f_{dim} + \frac{N_{plate}}{2} \right)^2 - \frac{2 \cdot P_u \cdot \left( e_{loading} + f_{dim} \right)}{q_{max}}} \qquad Y_{loading} = 0.327 \text{ in}$$

$$AISC DG#1 Eqn. 3.4.3$$

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$$T_u = 6.571 \ kip$$

 $T_u = 6.571 \text{ kip}$  AISC DG#1 Eqn. 3.4.2

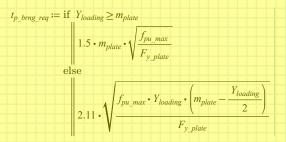
Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} \coloneqq \frac{N_{plate} - 0.8 \ OD_{post}}{2}$$

$$m_{plate} = 2.85$$
 in

Calculating minimum thickness based on bearing:



 $t_{p\_brng\_req} = 0.471 \ in$ 

AISC DG#1 Egns. 3.3.14a-2 &

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$$x_{ten} = 1.35 in$$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$$t_{p\_ten\_req} = 0.331$$
 in AISC DG#1 Eqn. 3.4.7a

Controlling minimum required base plate thickness:  $t_{p \ req} := \max (t_{p \ brng \ req}, t_{p \ ten \ req})$ 

$$t_{p\_req} := \max \left( t_{p\_brng\_req}, t_{p\_ten\_req} \right)$$

$$t_{p \ req} = 0.471 \ in$$

Check chosen plate thickness:

$$Plate\_Thick\_Check \coloneqq \text{if } t_p \ge t_{p\_req}$$
 \( \big| "Chosen plate thickness is adequate." \\ else \\ \big| "Need a thicker plate."

Plate Thick Check = "Chosen plate thickness is adequate."



Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size:

Min fillet weld size based on AISC Table J2-4

Chosen fillet weld size

Weld material:

 $F_{EXX} := 70 \text{ ksi}$ 

 $L_g := \pi \cdot OD_{post}$ 

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter:

Moment of Inertia of circular fillet weld:

Polar moment of Inertia of circular fillet weld:

Effective Length of Weld:

 $t_e := min\left(w \cdot \sin\left(45 \ deg\right), \frac{L_w}{4}\right)$   $t_e = 0.221 \ in$ 

 $L_w = 8.407 in$ 

Effective Throat Thickness:

Area of Weld:

 $A_w := L_w \cdot t_e$ 

 $L_g = 9.032$  in

 $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad I_w = 2.062 \text{ in}^4$ 

 $J_w := 2 \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad J_w = 4.124 \text{ in}^4$ 

17. Very thin annulus

AISC, Sect. J2, Pts. 2a & 2b

AISC, Sect. J2, Pts. 2a



 $A = 2\pi Rt$  $I_{\nu} = I_{\nu} = \pi R^3 t$  $r_{y} = r_{y} = 0.707R$ 

 $J = 2\pi r^3 t$ 

Determine design strength of weld:

Nominal strength of weld metal:

 $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ 

 $F_w = 31.5 \ ksi$ 

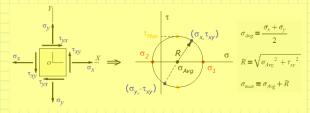
AISC, Tbl. J2.5

 $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$ 

Normal stress caused by bending moment:

Stress caused by shearing force:

Resultant stress in weld from loading:



Check of weld thickness:

Weld\_Design\_Check := if  $F_w \ge \sigma_{max}$ "Chosen weld size is sufficient." "Need bigger fillet weld."

Weld Design Check = "Chosen weld size is sufficient."



Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### **Anchor Bolt Connection Design**

Given:

Number of anchor bolts resisting loads:

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter:

Bolt area:

Bolt nominal yield stress strength:

Bolt nominal ultimate tensile stress strength:

Bolt embedment:

Output:

Tension anchor bolt spacing:

Bolt nominal tensile stress strength:

Bolt nominal shear stress strength:

Ultimate tension load on one anchor bolt:

Required shear stress on one bolt:

Bolt modified nominal tensile stress strength, modified for effects of shear stress:

Bolt factored tensile resistance:

Only one side's bolts resist tension or shear.

 $d_{ab} \coloneqq \frac{5}{8} in$ Plans

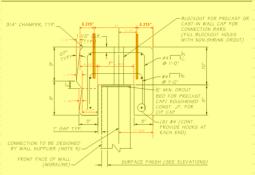
 $A_b := 0.307 \ in^2$ 

 $F_{v \ holt} := 36 \ ksi$ AISC Tbl. 2-3

 $F_{u \ bolt} := 58 \ ksi$ 

 $h_{ef} = 5$  in

AISC Tbl. 7-18



WALL CAP TYPICAL DETAIL

 $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{-t} - 1}$ 

 $F_{nt} := 0.75 \cdot F_{u \ bolt}$ 

 $F_{nt} = 43.5 \ ksi$ 

AISC Tbl. J3.2

AISC Tbl. J3.2, assuming threads within shear plane

 $F_{nv} := 0.40 \cdot F_{u \ bolt}$  $F_{nv} = 23.2 \ ksi$ 

 $T_{u\_ab} := \frac{T_u}{n_{ab}}$ 

 $f_{\rm v} = 1.14 \ ksi$ 

 $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_{v}\right)$ 

AISC Eqn. J3-3a

 $\phi R_{n \ bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$  $\phi R_{n \ bolt} = 10.016 \ kip$ 

Check of bolt tensile stress:

 $Bolt\_Tensile\_Check := if \phi R_{n \ bolt} \ge T_{u \ ab}$ "Bolt is satisfactory."

"Bolt is no good."

Bolt Tensile Check = "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor:

 $d_a := d_{ab}$   $d_a = 0.625$  in

Critical edge distance for adhesive anchors:

 $c_{ac} := 2 \ h_{ef}$   $c_{ac} = 10 \ in$ 

ACI 17.7.6

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: Group\_Tension\_Breakout\_Check := if  $s_1 \le 3 \cdot h_{ef}$ 

ACI 17.2.1.1

"Bolts act in group." "Bolts act singly."

Group\_Tension\_Breakout\_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

 $A_{Nco} := 9 \cdot h_{ef}^{2}$   $A_{Nco} = 225 \text{ in}^{2}$ 

ACI Eqn. 17.4.2.1c

Actual projected influence area for bolt(s):  $A_{Nc} := min\left(\left(c_{al} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_{1}, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ 

Concrete tension breakout strength coefficient:

 $k_c := 17$ 

Value of 17 for post-installed anchors, per ACI 17.4.2.2



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Basic concrete tension breakout strength for single anchor:

$$N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf \qquad N_b = 12.021 \ kip$$

Factor for eccentrically loaded anchor bolts:

$$\Psi_{-}$$
  $_{\nu}:=1.0$ 

 $\Psi_{ec\ N} := 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.4.2.4

Factor for anchor bolts near an edge:

$$\Psi_{ed_{N}} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{1.5 \cdot h_{ef}}\right)$$

$$\varPsi_{ed\_N} = 0.913$$

ACI Egn. 17.4.2.5b

Factor for anchor bolts in un-cracked concrete:

$$\Psi_{c N} := 1.4$$

 $\Psi_{c N} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.4.2.6

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\varPsi_{cp\_N} \coloneqq \min\left(1.0 \text{ , } \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right) \qquad \varPsi_{cp\_N} = 0.75 \qquad \text{ ACI Eqn. 17.4.2.7b}$$

$$\Psi_{cp\_N} = 0.75$$

Nominal concrete tension breakout strength:

$$\phi N_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b \qquad \phi N_{cbg} = 8.102 \ \textit{kip} \qquad \textit{ACI Eqn. 17.4.2.1b}$$

Check of concrete tension breakout failure:

$$\begin{aligned} \textit{Concrete\_Tension\_Breakout\_Check} \coloneqq & \text{if } \phi N_{cbg} \geq n_{ab} \cdot T_{u\_ab} \\ & \text{"Bolt is satisfactory."} \\ & \text{else} \\ & \text{"Bolt is no good."} \end{aligned}$$

Concrete\_Tension\_Breakout\_Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f_c}{2500 \ psi}\right)^{0.1} \cdot 2220 \ psi = 1512.441 \ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500 \ psi}\right)^{0.15} \cdot 2210 \ psi = 1541.429 \ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength:

$$\tau_{uncr} := min \left( \tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500} \right)$$

$$\tau_{uncr} = 1512.441 \ psi$$

Distance to edge of project influence area:

$$c_{Na} \coloneqq 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100 \ psi}}$$

$$c_{Na} = 7.329$$
 in

Check if anchor bolts act in group for bond failure:

Group\_Bond\_Failure\_Check := if 
$$s_1 \le 2$$
  $c_{Na}$ 

| "Bolts act in group

Group\_Bond\_Failure\_Check = "Bolts act in group."

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By: MJF Chk: RFA

On: 9/17/2021 On: 12/22/2021

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Nao} := (2 c_{Na})^{\frac{1}{2}}$$

 $A_{Nao} = 214.835 \text{ in}^2$ 

ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s):

$$A_{Na} := min\left(\left(c_{Na} + min\left(s_{1}, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$$

 $A_{Na} = 238.062 \text{ in}^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor:

$$N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$$

 $N_{ba} = 14.848 \ kip$ 

ACI Eqn. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec\ Na} := 1.0$$

Anchor bolts are not loaded eccentrically.

ACI 17.4.5.3

Factor for anchor bolts near an edge:

$$\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{c_{Na}}\right) \qquad \qquad \Psi_{ed\_Na} = 0.918$$

ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_Na} := min\left(1.0 \text{ , } \max\left(\frac{c_{al}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$$

$$\Psi_{cp\_Na} = 0.733$$
Wall caps are not under

ACI Eqn. 17.4.5.5b

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Bond\_Stress\_Check = "Bolt is satisfactory."

Nominal bond strength of the adhesive anchor(s): 
$$\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$$
  $\phi N_{ag} = 7.192 \text{ kip}$ 

ACI Eqn. 17.4.5.1b

Check of bolt bond stress failure:

$$Bond\_Stress\_Check \coloneqq \text{if } \phi N_{ag} \ge n_{ab} \cdot T_{u\_ab}$$
 \(\begin{aligned} "Bolt is satisfactory." \\ else \\ "Bolt is no good." \end{aligned}

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout:

Group Shear Breakout Check := if 
$$s_1 \le 3 \cdot c_{al}$$
 ACI 17.2.1.1 

"Bolts act in group." 

else 

"Bolts act singly."

Group Shear Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Vco} := 4.5 \cdot c_{al}^2$$

$$A_{Vco} = 127.122 \text{ in}^2$$

ACI Egn. 17.5.2.1c

Actual projected influence area for bolt(s): 
$$A_{Vc} := min\left(1.5 \cdot c_{al} \cdot \left(1.5 \cdot c_{al} + min\left(s_1, 3 \cdot c_{al}\right) + min\left(1.5 \cdot c_{al}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$$
  $A_{Vc} = 155.245 \ in^2$  ACI Fig. R17.5.2.1b

Load bearing length:

$$l_e := h_{ef}$$

$$l_e = 5$$
 in

ACI 17.5.2.2

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left[7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5 \ kip$$

ACI Eqns. 17.5.2.2a

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec}$$
  $_{V} := 1.0$ 

 $\Psi_{ec\ V} \coloneqq 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.5.2.5

Factor for anchor bolts near an edge:

$$\Psi_{ed_{\_}V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) \qquad \Psi_{ed_{\_}V} = 0.869$$

$$\Psi_{ed\_V} = 0.869$$

ACI Eqns. 17.5.2.6a & 17.5.2.6b

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{c,V} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.5.2.7



Chk: RFA

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Factor for small embedment

$$\varPsi_{h\_V} \coloneqq \min\left(1.0\;, \sqrt{\frac{1.5 \cdot c_{al}}{h_{ef}}}\right) \qquad \varPsi_{h\_V} = 1 \qquad \qquad \textit{ACI Eqn. 17.5.2.8}$$

Nominal concrete shear breakout strength:

$$\phi V_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b \qquad \phi V_{cbg} = 6.28 \text{ kip} \qquad \text{ACI Eqn. 17.5.2.1b}$$

Check of concrete shear breakout failure:

Concrete\_Shear\_Breakout\_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear:  $\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$ 

$$\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$$

 $\phi N_{cpg} = 7.192 \ kip$ 

ACI 17.5.3.1

ACI 17.5.3.1

Concrete pryout strength in shear coefficient:

Nominal concrete pryout strength of anchor(s) in shear:

$$\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$$

 $\phi V_{cpg} = 14.384 \ kip$ 

ACI Eqn. 17.5.3.1b

Check of concrete pryout strength in shear:

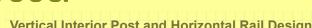
$$\begin{aligned} \textit{Concrete\_Shear\_Pryout\_Check} \coloneqq & \text{if } \phi \textit{V}_\textit{cpg} \geq \textit{V}_\textit{post\_u} \\ & \text{``Bolt is satisfactory.''} \\ & \text{else} \\ & \text{``Bolt is no good.''} \end{aligned}$$

Concrete Shear Pryout Check = "Bolt is satisfactory."



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vertical interior Post and Horizonta	i Raii Design			
Given:			Load Factors (AASHTO Tbl. 3.4.1-1):	
Post Height:	$H_{post} := 42$ in	Plans	PL Load Factor:	$\gamma_{PL} := 1.75$
Post and Rail Yield Strength:	$F_y := 50 \text{ ksi}$	ASTM F1043	DC Load Factor:	$\gamma_{DL} := 1.25$
Post and Rail Modulus of Elasticity:	$E_s := 29000 \ $ <i>ksi</i>		WS Load Factor:	$\gamma_{WS} := 1.00$
Post and Rail Ultimate Strength:	$F_u := 58 \text{ ksi}$			
Proportion for ACTM F1042 IC 1 7/9" Pine	for Interior Deete		Resistance Factors:	
Properties for ASTM F1043 IC 1-7/8" Pipe			Steel Flexure (AASHTO 6.5.4.2):	$\phi_f := 1.00$
Post OD:	$OD_{post} := 1.900 \ in$	1	Steel Shear (AASHTO 6.5.4.2):	$\phi_{v} := 1.00$
Post Thickness:	$t_{post} \coloneqq 0.12 \ \textit{in}$		Tension, Yielding in Gross Section:	$\phi_y := 0.95$
Properties for ASTM F1043 IC 1-5/8" Pipe	for Rails		Bending (AISC F1):	$\phi_b := 0.90$
Rail OD:	$OD_{rail} := 1.66$ in		Shear (AISC G1):	$\phi_{v\_AISC} := 0.90$
Rail Thickness:	$t_{rail} := 0.111 \ in$		Bearing (AISC DG#1):	$\phi_{brg} := 0.60$
Design Deint Live Load	D 200 # 6	4404704000	Fillet Weld (AISC Tbl. J2.5):	$\phi_{fw} := 0.75$
Design Point Live Load	$P_{LL} \coloneqq 200 \; lbf$	AASHTO 13.8.2	Bolts (AISC J3.6 & J3.7):	$\phi_{ab} := 0.75$
Design Uniform Live Load	$w_{LL} \coloneqq 0 \ plf$	AASHTO 13.8.2	Adhesive Anchor Bolts (ACI 17.3.3,	
Post spacing:	$L_{spc} := 8  ft$	Plans	Condition B, Category 1):	$\phi_{adh} := 0.65$
Weight of chain link fence:	$f_{clf} := 0.48 \; psf$			
Design wind load from chain link fence:	$f_{wind} := 15 \ psf$	AASHTO 13.8.2	Stool weight density:	

A CTM E4	ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe															
Trade	Decima	I O.D.	Pipe \	wall			Sect	tion	Ш	Min. Yield		П	Max Bending	Calcula	ated Load (I	os)
Reference	Equiva	alent	Thickn	iess	Weig	ht	Modu	ulus	X	Stren	gth	=	Moment	10' Free	Canti	lever
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches <sup>3</sup>	(mm³)	Х	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	Х	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	Х	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	X	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	E	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	х	50000	345	Ξ	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	E	114295	3810	5486	1587
									П			т				

Steel weight density:

#### Output:

ŀ	0	St	S	ec	tio	n	РΙ	O	per	tie	S:	

Post inside diameter:	$ID_{post} := OD_{post} - 2 \cdot t_{post}$	$ID_{post} = 1.66$ in
Post Area:	$A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$	$A_{post} = 0.671 \text{ in}^2$
Post Unit Weight:	$w_{post} \coloneqq \gamma_{steel} \cdot A_{post}$	$w_{post} = 2.283 \ plf$
Post centroid:	$c_{post} := 0.5 \cdot OD_{post}$	$c_{post} = 0.95 \ in$
Post Moment of Inertial:	$I_{post} \coloneqq 0.049087 \cdot \left(OD_{post}^{ 4} - ID_{post}^{ 4}\right)$	$I_{post} = 0.267 \ in^4$
Post Section Modulus:	$S_{post} := \frac{I_{post}}{c_{post}}$	$S_{post} = 0.281 \text{ in}^3$
Post Plastic Section Modulus:	$Z_{post} := \frac{OD_{post}^{3} - ID_{post}^{3}}{6}$	$Z_{post} = 0.381 \text{ in}^3$
Rail Section Properties:		
Rail inside diameter:	$ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$	$ID_{rail} = 1.438 \ in$
Rail Area:	$A_{rail} \coloneqq 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$	$A_{rail} = 0.54 \ in^2$
Rail Unit Weight:	$w_{rail} \coloneqq \gamma_{steel} \cdot A_{rail}$	$w_{rail} = 1.838  plf$
Rail centroid:	$c_{rail} := 0.5 \cdot OD_{rail}$	$c_{rail} = 0.83$ in
Rail Moment of Inertial:	$I_{rail} := 0.049087 \cdot \left(OD_{rail}^{4} - ID_{rail}^{4}\right)$	$I_{rail} = 0.163 \text{ in}^4$
Rail Section Modulus:	$S_{rail} := \frac{I_{rail}}{c_{rail}}$	$S_{rail} = 0.196 \ in^3$
Rail Plastic Section Modulus:	$Z_{rail} := \frac{OD_{rail}^3 - ID_{rail}^3}{6}$	$Z_{rail} = 0.267 \text{ in}^3$



 $\gamma_{steel} := 490 \ pcf$ 

$$A = \frac{\pi(d - d_1)}{4} = .785398 (d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087 (d^4 - d_1^4)$$

$$S = \frac{\pi(d^4 - d_1^4)}{32 d} = .098175 \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$



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 $\phi M_n = 19.039 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

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Post concentrated live load applied at top rail:	$P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.2 \text{ kip}$	$P_{post\_LL} = 0.2 \text{ kip}$	AASHTO Eqn. 13.8.2-1
Post moment loading from live load:	$M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 8400 \ lbf \cdot in$	$M_{post\_LL} = 8400 \; lbf \cdot in$	Post treated as cantilevered beam
Post shear from live load:	$V_{post\_LL} \coloneqq P_{post\_LL}$	$V_{post\_LL} = 0.2 \text{ kip}$	
Rail moment from live load applied:	$M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $H_{post}  L_{2}$	$M_{rail\_LL} = 4800 \; lbf \cdot in$	Rail treated as simply supported beam with vertical and horizontal live loads combined into
Rail moment from dead load:	$M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{2}}{8}$	$M_{rail\_DL} = 257.093 \ lbf \cdot in$	resultant direction.
Rail shear from live load:	$V_{rail\_LL} \coloneqq \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$	$V_{rail\_LL} = 0.1 \text{ kip}$	
Rail shear from dead load:	$V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$	$V_{rail\_DL} = 0.011 \ \textit{kip}$	AASHTO load factors
Factored Shear Load on Post:	$V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$	$V_{post\_u} = 0.35 \text{ kip}$	used instead of ASCE load factors found in AISC and ACI. This is
Factored Moment Load on Post:	$M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$	$M_{post\_u} = 14700 \; \textit{lbf} \cdot \textit{in}$	acceptable as it is more conservative.
Factored Shear Load on Rail:	$V_{rail\_u} \coloneqq \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$	$V_{rail\_u} = 0.188 \text{ kip}$	Vertical dead load was combined directly with live load resultant since
Factored Moment Load on Rail:	$M_{rail\_u} \coloneqq \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$	$M_{rail\_u} = 8721.366 \ lbf \cdot in$	it was so small compared to the live load.
Post Analysis: Following AASHTO 6.12.1.2.3c for Shear Design Gross Area:	$A_g \coloneqq A_{post}$	$A_g = 0.671 \text{ in}^2$	
Distance from Max to 0 Shear:	$L_{v} := H_{post}$	$L_{v} = 42 \text{ in}$	
Critical Strength for Shear: $F_{cr} \coloneqq \min \left( 0.58 \cdot F_{y} \right)$	$\max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}}} \left( \frac{OD_{post}}{t_{post}} \right)^{\frac{5}{4}} \cdot , \frac{0.78 \cdot E_s}{\left( \frac{OD_{post}}{t_{post}} \right)^{\frac{3}{2}}} \right) \right)$	F <sub>cr</sub> = 29 <b>ksi</b> AASH1 & 6.12.	TO Eqns. 6.12.1.2.3c-2 1.2.3c-3
Factored nominal shear resistance:	$\phi V_n := \phi_v \cdot 0.5 \ F_{cr} \cdot A_g$		HTO Eqn. 6.12.1.2.3c-1
Post Shear Check:	$\frac{\phi V_n}{V_{post\_u}} = 27.8$ Post_Shear_C	$heck := if \frac{\phi V_n}{V_{post\_u}} \ge 1.0$   "Post shear strengthelse"    "Post is not satisfaction."	
	Post_Shear_C	heck = "Post shear strength is s	satisfactory."
Following AASHTO 6.12.2.2.3 for Flexure Design			
Check of Noncompact Section: Check_Compa	$act := if \frac{OD_{post}}{t_{post}} \le \frac{0.07 \cdot E_s}{F_y}$	D/t does no plastic mod 6.12.2.2.3-1	O 6.12.2.2.3, as long t exceed 0.07E/Fy, ulus and equation I may be used.
Check_Compo	act = "Section is compact. Local buckling does i		
E-tININIPI		/1/	

 $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ 

Factored Nominal Moment Resistance:



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Post\_Flex\_Check := if  $\frac{\phi M_n}{M_{post\_u}} \ge 1.0$ "Post flexural strength is satisfactory." Post Flexural Check: "Post is not satisfactory."

Post Flex Check = "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area:

 $A_g = 0.54 \text{ in}^3$ 

Distance from Max to 0 Shear:

Critical Strength for Shear:

 $F_{cr} := min \left( 0.58 \cdot F_y, \max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}} , \frac{0.78 \cdot E_s}{\sqrt{\frac{OD_{rail}}{1000}}} \right) \right)$ 

AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance:

 $\phi V_n = 7.832$  kip AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check:

 $\frac{\phi V_n}{V_{rail\_u}} = 41.575$   $Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0$ "Rail shear strength is satisfactory." "Rail is not satisfactory."

Rail\_Shear\_Check = "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

Check\_Compact := if  $\frac{OD_{rail}}{t_{rail}} \le \frac{0.07 \cdot E_s}{F_y}$ "Section is compact. Local buckling does not apply." "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

Check\_Compact = "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance:

 $\phi M_n := \phi_f \cdot F_v \cdot Z_{rail}$ 

 $\phi M_n = 13.339 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

 $\frac{\phi M_n}{M_{rail\ u}} = 1.53$ 

Rail\_Flex\_Check := if  $\frac{\phi M_n}{M_{rail\_u}} \ge 1.0$ | "Rail flexural strength is satisfactory."

else "Rail is not satisfactory."

Rail Flex Check = "Rail flexural strength is satisfactory."



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Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post:

$$w_{post\ wind} := f_{wind} \cdot L_{spc}$$

$$w_{post\ wind} = 120\ plf$$

Design moment from wind on post:

$$M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$$

$$M_{post\_wind\_u} = 8820$$
 lbf • in

$$V_{post\ wind\ u} := \gamma_{WS} \cdot w_{post\ wind} \cdot H_{post}$$

$$V_{post\ wind\ u} = 0.42\ kip$$

$$V_{post\ u} = 0.35\ \textit{kip}$$
 <- LL controls

 $M_{post\ u} = 14700\ \textit{lbf} \cdot \textit{in} < - LL \text{ controls}$ 

Uniform wind on rail:

$$w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$$

$$w_{rail\_wind} = 26.25 \ plf$$

Design moment from wind on rail:

$$M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$$

$$M_{rail\_wind\_u} = 2520 \ \textit{lbf} \cdot \textit{in}$$

Design shear from wind on rail: 
$$V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$$

 $M_{rail\ u} = 8721.366\ lbf \cdot in <- LL controls$ 

$$V_{rail\ u} = 0.188$$
 kip

 $V_{rail\_wind\_u} = 0.105$  kip

<- LL controls



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#### Base Plate Design - Line Post w/ Axial Compression

Plans

Given:

Cap width:  $W_{cap} := 15.63 \ in$ Distance from post to end of cap:  $L_{end} := 96$  in

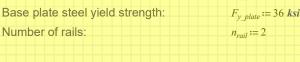
Plate thickness:  $t_n := 0.5 \ in$ 

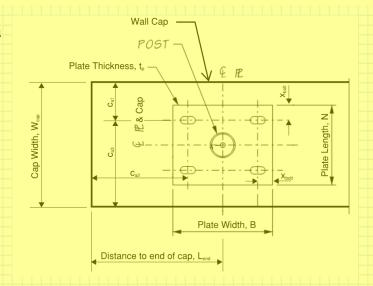
 $N_{plate} := 8 in$ Plate length (perpendicular to fence):

Plate width (parallel to fence):  $B_{nlate} := 10$  in

Compressive Strength of Concrete:  $f_c := 4 \text{ ksi}$ 

Side clearance to anchor bolts:  $x_{bolt} := 1.5 in$ 





#### Output:

Plate Area:

$$A_{plate} := N_{plate} \cdot B_{plate}$$

$$A_{plate} = 80 \text{ in}^2$$

 $c_{al} = 5.315$  in

Distance from bolt to near face of cap:

$$c_{al} := \frac{1}{2} \left( W_{cap} - N_{plate} \right) + x_{bolt}$$

$$c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$$

$$c_{a2} = 92.5$$
 in

Distance from outside bolt to end of cap: Distance from bolt to far face of cap:

$$c_{a3} \coloneqq W_{cap} - c_{a1}$$

$$c_{a3} = 10.315$$
 in

Conservatively setting

Bearing Area taken to Be Same as Plate Area:

$$A_{bearing} := A_{plate}$$

 $A_{bearing} = 80 \text{ in}^2$ bearing area to the same as the plate.

Max allowed bearing pressure:

$$f_{pu\_max} := \phi_{brg} \cdot min \left( 0.85 \cdot f_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}} \right), 1.7 \cdot f_c$$

$$f_{pu\_max} = 2.04 \text{ ksi}$$
 ACI Tbl. 14.5.6.1

Max allowed bearing pressure line:

$$q_{max} := f_{pu\_max} \cdot B_{plate}$$

$$P_{post\ DL} := w_{post} \cdot H_{post}$$

 $q_{max} = (2.448 \cdot 10^5) \frac{lbf}{ft}$  $P_{post DL} = 0.008 \ kip$ 

Post dead load on plate Rail dead load on plat:

$$P_{rail\ DL} := n_{rail} \cdot 2 \cdot V_{rail\ DL}$$

 $P_{rail\ DL} = 0.043 \ kip$ 

Factored vertical load on plate:

$$P_u := \gamma_{DL} \cdot \left( P_{post \ DL} + P_{rail \ DL} \right)$$

 $P_u = 0.064 \ kip$ 

Minimum length of area of bearing:

$$Y_{min} := \frac{P_u}{q_{max}}$$

 $Y_{min} = 0.003$  in AISC DG#1 Eqn. 3.3.3

Critical eccentricity distance:

$$e_{crit} \coloneqq \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$$

 $e_{crit} = 3.998 in$ AISC DG#1 Eqn. 3.3.7

Eccentricity of loading:

$$e_{loading} \coloneqq \frac{M_{post\_u}}{P_u}$$

 $e_{loading} = 231.31$  in

AISC DG#1 Eqn. 3.3.6

Small moment check:

$$Small\_Moment\_Check := if_{loading} \le e_{crit}$$

"Moment is small, no need for anchor bolts."

"Moment is large, need anchor bolts."

Small Moment Check = "Moment is large, need anchor bolts."

Distance from bolt to center of post:

$$f_{dim} := \frac{N_{plate}}{2} - x_{bolt} \qquad \qquad f_{dim} = 2.5 \text{ in}$$

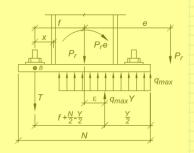


Fig. 3.4.1. Base plate with large moment.



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Plate dimension check

$$\begin{aligned} \textit{Plate\_Dim\_Check} \coloneqq & \text{if } \left( f_{\textit{dim}} + \frac{N_{\textit{plate}}}{2} \right)^2 \geq \frac{2 \cdot P_u \cdot \left( e_{\textit{loading}} + f_{\textit{dim}} \right)}{q_{\textit{max}}} \\ & \parallel \text{``Plate dimensions are OK.''} \\ & \text{else} \\ & \parallel \text{``Plate needs to be longer and/or wider.''} \end{aligned}$$

Plate Dim Check = "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} \coloneqq \left( f_{dim} + \frac{N_{plate}}{2} \right) - \sqrt{\left( f_{dim} + \frac{N_{plate}}{2} \right)^2 - \frac{2 \cdot P_u \cdot \left( e_{loading} + f_{dim} \right)}{q_{max}}} \qquad Y_{loading} = 0.113 \ \textit{in}$$

$$AISC \ DG\#1 \ Eqn. \ 3.4.3$$

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$$T_u = 2.242 \ kip$$

 $T_u = 2.242 \text{ kip}$  AISC DG#1 Eqn. 3.4.2

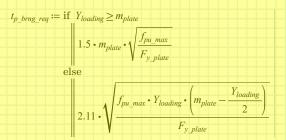
Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} \coloneqq \frac{N_{plate} - 0.8 \ OD_{post}}{2}$$

$$m_{plate} = 3.24$$
 in

Calculating minimum thickness based on bearing:



 $t_{p\_brng\_reg} = 0.301 \ in$ 

AISC DG#1 Egns. 3.3.14a-2 &

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$$x_{ten} = 1.74 \ in$$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$$p_{ten\_req} = 0.22 in$$
 AISC DG#1 Eqn. 3.4.7a

Controlling minimum required base plate thickness:  $t_{p \ req} := \max (t_{p \ brng \ req}, t_{p \ ten \ req})$ 

$$t_{p\_req} := \max \left( t_{p\_brng\_req}, t_{p\_ten\_req} \right)$$

$$t_{p \ req} = 0.301 \ in$$

Check chosen plate thickness:

$$\begin{aligned} \textit{Plate\_Thick\_Check} &\coloneqq \text{if } t_p \geq t_{p\_req} \\ &\parallel \text{``Chosen plate thickness is adequate.''} \\ &= \text{else} \\ &\parallel \text{``Need a thicker plate.''} \end{aligned}$$

Plate Thick Check = "Chosen plate thickness is adequate."



Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size:

Min fillet weld size based on AISC Table J2-4

Chosen fillet weld size

Weld material:

 $F_{EXX} := 70 \text{ ksi}$ 

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter:

Effective Length of Weld:

 $L_g = 5.969$  in

 $L_w = 5.344$  in

Effective Throat Thickness:

 $t_e := min\left(w \cdot \sin\left(45 \ deg\right), \frac{L_w}{4}\right)$   $t_e = 0.221 \ in$ 

 $L_g := \pi \cdot OD_{post}$ 

AISC, Sect. J2, Pts. 2a & 2b

AISC, Sect. J2, Pts. 2a

Area of Weld:

Moment of Inertia of circular fillet weld:

Polar moment of Inertia of circular fillet weld:

$$\begin{split} A_w &\coloneqq L_w \cdot t_e & A_w = 1.181 \ \textit{in}^2 \\ I_w &\coloneqq \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e & I_w = 0.595 \ \textit{in}^4 \end{split}$$

 $J_w := 2 \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad J_w = 1.19 \text{ in}^4$ 

17. Very thin annulus



 $A = 2\pi Rt$  $I_{\nu} = I_{\nu} = \pi R^3 t$  $r_{y} = r_{y} = 0.707R$ 

 $J = 2\pi r^3 t$ 

Determine design strength of weld:

Nominal strength of weld metal:

 $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ 

 $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I}$ 

 $F_w = 31.5 \ ksi$ 

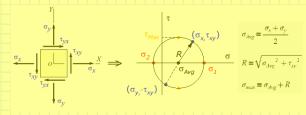
AISC, Tbl. J2.5

Normal stress caused by bending moment:

Stress caused by shearing force:

 $\sigma_b = 23.463 \text{ ksi}$   $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$ 

Resultant stress in weld from loading:



Check of weld thickness:

Weld\_Design\_Check := if  $F_w \ge \sigma_{max}$ 

"Chosen weld size is sufficient."

"Need bigger fillet weld."

Weld Design Check = "Chosen weld size is sufficient."



By: MJF Chk: RFA

On: 9/17/2021 On: 12/22/2021

#### **Anchor Bolt Connection Design**

Given:

Number of anchor bolts resisting loads:

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter:

Bolt area:

Bolt nominal yield stress strength:

Bolt nominal ultimate tensile stress strength:

Bolt embedment:

Output:

Tension anchor bolt spacing:

Bolt nominal tensile stress strength:

Bolt nominal shear stress strength:

Ultimate tension load on one anchor bolt:

Required shear stress on one bolt:

Bolt modified nominal tensile stress strength, modified for effects of shear stress:

Bolt factored tensile resistance:

Only one side's bolts resist tension or shear.

 $d_{ab} \coloneqq \frac{5}{8} in$ Plans

 $A_b := 0.307 \ in^2$ 

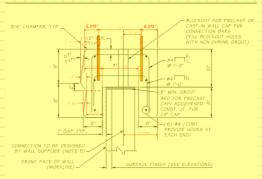
AISC Tbl. 7-18

 $F_{v bolt} := 36$  ksi

AISC Tbl. 2-3

 $F_{u \ bolt} := 58 \ ksi$ 

 $h_{ef} := 5$  in



WALL CAP TYPICAL DETAIL

 $s_I := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1} \qquad s_I = 7 \text{ in}$ 

AISC Tbl. J3.2

AISC Tbl. J3.2, assuming threads within shear plane

 $F_{nv} := 0.40 \cdot F_{u \ bolt}$  $T_{u\ ab} = 1.121\ kip$ 

 $f_{\rm v} = 0.57 \ {\it ksi}$ 

 $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$   $F_{nt}' = 43.5 \text{ ksi}$  AISC Eqn. J3-3a

 $\phi R_{n \ bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$   $\phi R_{n \ bolt} = 10.016 \ kip$ 

AISC Eqn. J3-2

ACI 17.2.1.1

*Bolt\_Tensile\_Check* = "Bolt is satisfactory."

Check of bolt tensile stress:

Bolt\_Tensile\_Check := if  $\phi R_{n \ bolt} \ge T_{u \ ab}$ 

"Bolt is satisfactory."

"Bolt is no good."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor:

 $d_a := d_{ab}$ 

 $d_a = 0.625$  in

Critical edge distance for adhesive anchors:

 $c_{ac} \coloneqq 2 h_{ef}$   $c_{ac} = 10 in$ 

ACI 17.7.6

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout:

*Group Tension Breakout Check* := if  $s_1 \le 3 \cdot h_{ef}$ 

"Bolts act in group."

"Bolts act singly."

Group Tension Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

 $A_{Nco} := 9 \cdot h_{ef}^2$ 

 $A_{Nco} = 225 \ in^2$ 

ACI Eqn. 17.4.2.1c

Actual projected influence area for bolt(s):  $A_{Nc} := min\left(\left(c_{al} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_{1}, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$   $A_{Nc} = 281.93 \ \text{in}^{2} \quad A_{Cl} = 281.93 \ \text{in}^{2} \quad$ 

Concrete tension breakout strength coefficient:

Value of 17 for post-installed anchors, per ACI 17.4.2.2



By: MJF Chk: RFA

On: 9/17/2021 On: 12/22/2021

Basic concrete tension breakout strength for single anchor:

$$N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$$
  $N_b = 12.021 \ kip$  ACI Eqn. 17.4.2.2a

Factor for eccentrically loaded anchor bolts:

 $\Psi_{ec\ N} := 1.0$ 

Anchor bolts are not loaded eccentrically.

ACI 17424

Factor for anchor bolts near an edge:

$$\Psi_{ed\_N} \coloneqq \min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{1.5 \cdot h_{ef}}\right)$$

 $\Psi_{ed\ N} = 0.913$  ACI Eqn. 17.4.2.5b

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{c,N} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.4.2.6

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_N} := min\left(1.0 \text{ , max}\left(\frac{c_{al}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right) \qquad \Psi_{cp\_N} = 0.75 \qquad \text{ACI Eqn. 17.4.2.7b}$$

Nominal concrete tension breakout strength:

$$\phi N_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b \qquad \phi N_{cbg} = 9.382 \text{ kip} \qquad \text{ACI Eqn. 17.4.2.1b}$$

Check of concrete tension breakout failure:

$$Concrete\_Tension\_Breakout\_Check \coloneqq \text{if } \phi N_{cbg} \ge n_{ab} \cdot T_{u\_ab}$$

$$\parallel \text{``Bolt is satisfactory.''}$$

$$\text{else}$$

$$\parallel \text{``Bolt is no good.''}$$

Concrete Tension Breakout Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500 \ psi}\right)^{0.1} \cdot 2220 \ psi = 1512.441 \ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500 \text{ psi}}\right)^{0.15} \cdot 2210 \text{ psi} = 1541.429 \text{ psi}$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below

Minimum bond stress strength:

$$\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right) \qquad \qquad \tau_{uncr} = 1512.441 \ \textit{psi}$$

$$\tau_{uncr} = 1512.441 \ psi$$

Distance to edge of project influence area:

$$c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100 \text{ nsi}}}$$

$$c_{Na} = 7.329$$
 in

ACI Egn. 17.4.5.1d

Check if anchor bolts act in group for bond failure:

$$Group\_Bond\_Failure\_Check := if s_1 \le 2 c_{Na}$$

"Bolts act in group."

Group Bond Failure Check = "Bolts act in group."



By: MJF Chk: RFA

On: 9/17/2021 On: 12/22/2021

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Nao} := (2 c_{Na})$$

 $A_{Nao} = 214.835 \text{ in}^2$ 

ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s):

$$A_{Na} := min\left(\left(c_{Na} + min\left(s_{1}, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$$

 $A_{Na} = 273.826 \text{ in}^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor:

$$N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$$

 $N_{ba} = 14.848 \ kip$ 

ACI Ean. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts:

 $\Psi_{ec,Na} := 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.4.5.3

Factor for anchor bolts near an edge:

$$\Psi_{ed\_Na} := min \left( 1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{c_{Na}} \right)$$

 $\Psi_{ed\ Na} = 0.918$ 

ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_Na} := min\left(1.0 , \max\left(\frac{c_{al}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$$

 $\Psi_{cp\_Na}=0.733$  ACI Eqn. 17.4.5.5b Wall caps are not under load, and per the wall cap design, vice moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s):

$$\phi N_{ag} \coloneqq \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba} \qquad \phi N_{ag} = 8.272 \ \textit{kip}$$

Check of bolt bond stress failure:

$$Bond\_Stress\_Check \coloneqq \text{if } \phi N_{ag} \geq n_{ab} \cdot T_{u\_ab}$$
 \text{"Bolt is satisfactory."} else \text{"Bolt is no good."}

Bond Stress Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout:

Group\_Shear\_Breakout\_Check := if 
$$s_1 \le 3 \cdot c_{al}$$
 "Bolts act in group."

"Bolts act singly."

Group Shear Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Vco} := 4.5 \cdot c_{al}^{2}$$

$$A_{Vco} = 127.122 \ in^2$$

ACI Eqn. 17.5.2.1c

ACI 17.2.1.1

Actual projected influence area for bolt(s): 
$$A_{Vc} := \min\left(1.5 \cdot c_{al} \cdot \left(1.5 \cdot c_{al} + \min\left(s_1, 3 \cdot c_{al}\right) + \min\left(1.5 \cdot c_{al}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right) \quad A_{Vc} = 182.929 \text{ in}^2 \quad A_{Cl} \text{ Fig. R17.5.2.1b}$$

 $l_o = 5$  in ACI 17.5.2.2

Load bearing length:

$$l_e := h_{ef}$$

$$l_e = 5$$
 in

Basic concrete breakout strength in shear for single anchor:

$$V_b \coloneqq \min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5 \ \textit{kip}$$

Concrete is not light weight;

ACI Eqns. 17.5.2.2a

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec}$$
  $_{V} := 1.0$ 

 $\Psi_{ec\ V} \coloneqq 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.5.2.5

Factor for anchor bolts near an edge:

$$\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$$

$$\Psi_{ed_{V}} = 1$$

ACI Egns. 17.5.2.6a

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{c,V} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.5.2.7



Chk: RFA

On: 9/17/2021 On: 12/22/2021

Factor for small embedment

$$\Psi_{h\_V} := min\left(1.0 \;, \sqrt{\frac{1.5 \cdot c_{al}}{h_{ef}}}\right) \qquad \Psi_{h\_V} = 1$$
 ACI Eqn. 17.5.2.8

Nominal concrete shear breakout strength:

$$\phi V_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b \qquad \qquad \phi V_{cbg} = 8.512 \ \textit{kip} \qquad \text{ACI Eqn. 17.5.2.1b}$$

Check of concrete shear breakout failure:

Concrete\_Shear\_Breakout\_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear:  $\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$ 

$$\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$$

 $\phi N_{cpg} = 8.272 \text{ kip}$ 

ACI 17.5.3.1

ACI 17.5.3.1

Concrete pryout strength in shear coefficient:

Nominal concrete pryout strength of anchor(s) in shear:

$$\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$$

 $\phi V_{cpg} = 16.545 \ kip$ 

ACI Eqn. 17.5.3.1b

Check of concrete pryout strength in shear:

Concrete\_Shear\_Pryout\_Check = "Bolt is satisfactory."





By: MJF On: 9/17/2021 Chk: RFA On: 12/22/2021

 $\gamma_{steel} := 490 \ pcf$ 

Vertical	Interior	Post and	d Horizontal	Rail Design	n
					Т
0.					

Given:			Load Factors (AASHTO Tbl. 3.4.1-1):	
Post Height:	$H_{post} := 66$ in	Plans	PL Load Factor:	$\gamma_{PL} := 1.75$
Step Height:	$H_{step} := 24$ in		DC Load Factor:	$\gamma_{DL} := 1.25$
Post and Rail Yield Strength:	$F_y := 50 \ \textit{ksi}$	ASTM F1043	WS Load Factor:	$\gamma_{WS} := 1.00$
Post and Rail Modulus of Elasticity:	$E_s := 29000 \ ksi$			
Post and Rail Ultimate Strength:	$F_u := 58 \ \textit{ksi}$		Resistance Factors:	
Draw artists for ACTM E4042 IC 2 2/01 Direct	for Interior Docto		Steel Flexure (AASHTO 6.5.4.2):	$\phi_f := 1.00$
Properties for ASTM F1043 IC 2-3/8" Pipe	e for interior Posts		Steel Shear (AASHTO 6.5.4.2):	$\phi_{v} := 1.00$
Post OD:	$OD_{post} := 2.375 i r$	1	Tension, Yielding in Gross Section:	$\phi_{v} \coloneqq 0.95$
Post Thickness:	$t_{post} := 0.13 \ in$			
			Bending (AISC F1):	$\phi_b := 0.90$
Properties for ASTM F1043 IC 1-5/8" Pipe	e for Rails		Shear (AISC G1):	$\phi_{v\_AISC} := 0.90$
Rail OD:	$OD_{rail} := 1.66 \ in$		Bearing (AISC DG#1):	$\phi_{brg} := 0.60$
Rail Thickness:	$t_{rail} := 0.111 \ in$		Fillet Weld (AISC Tbl. J2.5):	$\phi_{fw} := 0.75$
			Bolts (AISC J3.6 & J3.7):	$\phi_{ab} := 0.75$
Design Point Live Load	$P_{LL} := 200 \ lbf$	AASHTO 13.8.2	Adhesive Anchor Bolts (ACI 17.3.3,	r uo
Design Uniform Live Load	$w_{LL} := 0 \ plf$	AASHTO 13.8.2	Condition B, Category 1):	$\phi_{adh} := 0.65$
Post spacing:	$L_{spc} := 8  \text{ft}$	Plans		
Weight of chain link fence:	$f_{out} = 0.48 \ psf$			

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe																
Trade	Decima	I O.D.	Pipe v	wall			Section		П	Min. Yield		П	Max Bending	Calculated Load (lbs)		
Reference	Equiva	alent	Thickn	ess	Weig	ht	Modu	ulus	X	Stren	gth	=	Moment	10' Free	Canti	lever
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches <sup>3</sup>	(mm³)	X	psi	(Mpa)	=	Lb.ln.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	Х	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	х	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	х	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	Х	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	х	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	Х	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	I	114295	3810	5486	1587
									П			П				

AASHTO 13.8.2

Steel weight density:

 $c_{rail} = 0.83$  in

 $I_{rail} = 0.163 \text{ in}^4$ 

 $S_{rail} = 0.196 \text{ in}^3$ 

 $Z_{rail} = 0.267 \text{ in}^3$ 

#### Output:

Rail centroid:

Rail Moment of Inertial:

Rail Section Modulus:

Rail Plastic Section Modulus:

Post Section Properties:

Design wind load from chain link fence:

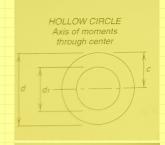
Post inside diameter:	$ID_{post} := OD_{post} - 2 \cdot t_{post}$	$ID_{post} = 2.115 \ in$
Post Area:	$A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$	$A_{post} = 0.917 \text{ in}^2$
Post Unit Weight:	$w_{post} := \gamma_{steel} \cdot A_{post}$	$w_{post} = 3.12  plf$
Post centroid:	$c_{post} := 0.5 \cdot OD_{post}$	$c_{post} = 1.188 \ in$
Post Moment of Inertial:	$I_{post} := 0.049087 \cdot \left(OD_{post}^{4} - ID_{post}^{4}\right)$	$I_{post} = 0.58 \text{ in}^4$
Post Section Modulus:	$S_{post} \coloneqq \frac{I_{post}}{c_{post}}$	$S_{post} = 0.488 \text{ in}^3$
Post Plastic Section Modulus:	$Z_{post} := \frac{OD_{post}^{3} - ID_{post}^{3}}{6}$	$Z_{post} = 0.656 \text{ in}^3$
Rail Section Properties:		
Rail inside diameter:	$ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$	$ID_{rail} = 1.438 \ in$
Rail Area:	$A_{rail} \coloneqq 0.785398 \cdot \left( OD_{rail}^2 - ID_{rail}^2 \right)$	$A_{rail} = 0.54 \ in^2$
Rail Unit Weight:	$W_{rail} := \gamma_{steel} \cdot A_{rail}$	$w_{rail} = 1.838  plf$

 $c_{rail} := 0.5 \cdot OD_{rail}$ 

 $Z_{rail} := \frac{OD_{rail}^3 - ID_{rail}^3}{6}$ 

 $I_{rail} := 0.049087 \cdot \left(OD_{rail}^{4} - ID_{rail}^{4}\right)$ 

 $f_{wind} := 15 \ psf$ 



$$A = \frac{\pi(d^{4} - d_{1}^{4})}{4} = .785398 (d^{2} - d_{1}^{2})$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^{4} - d_{1}^{4})}{64} = .049087 (d^{4} - d_{1}^{4})$$

$$S = \frac{\pi(d^{4} - d_{1}^{4})}{32 d} = .098175 \frac{d^{4} - d_{1}^{4}}{d}$$

$$I = \frac{\sqrt{d^{2} + d_{1}^{2}}}{4}$$

$$I = \frac{d^{3} - d_{1}^{3}}{6}$$



By: MJF Chk: RFA On: 9/17/2021 On: 12/22/2021

Post concentrated live load applied at high to	op rail: $P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.2 \text{ kip}$	$P_{post\_LL\_H} = 0.2 \ \emph{kip}$ AASHTO Eqn. 13.8.2-1, modified for split top rails
Post concentrated live load applied at low to	p rail: $P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0 \text{ kip}$	$P_{post\_LL\_L} = 0  kip$
Post moment loading from live load: $M_{post}$	$_{LL} \coloneqq P_{post\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot \left(H_{post} - H_{step}\right)$	$M_{post\_LL} = 13200 \; \textit{lbf} \cdot \textit{in}$ Post treated as cantilevered beam
Post shear from live load:	$V_{post\_LL} := P_{post\_LL\_H} + P_{post\_LL\_L}$	$V_{post\_LL} = 0.2 \text{ kip}$
Rail moment from live load applied:	$M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $H_{} \cdot \frac{1}{2} \cdot$	Rail treated as simply supported beam with vertical and horizontal live loads combined into
Rail moment from dead load:	$M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{8}$	resultant direction. $M_{rail\_DL} = 303.173 \; \textit{lbf} \cdot \textit{in}$
Rail shear from live load:	$V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$	
Rail shear from dead load:	$V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$	
Factored Shear Load on Post:	$V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$	$V_{post\_u} = 0.35$ kip  AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is
Factored Moment Load on Post:	$M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$	$M_{post\_u} = 23100 \ lbf \cdot in$ acceptable as it is more conservative.
Factored Shear Load on Rail:	$V_{rail\_u} \coloneqq \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$	$V_{rail\_u} = 0.191$ kip Vertical dead load was combined directly with live load resultant since
Factored Moment Load on Rail:	$M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$	$M_{rail\_u} = 8778.966$ <b>lbf</b> • in it was so small compared to the live load.
Post Analysis: Following AASHTO 6.12.1.2.3c for Shear De Gross Area:	esign: $A_g \coloneqq A_{post}$	$A_g = 0.917 \text{ in}^2$
Distance from Max to 0 Shear:	$L_{v} \coloneqq H_{post}$	$L_v = 66$ in
Critical Strength for Shear: $F_{cr} := min \left(0.53\right)$	$8 \cdot F_{y}, \max \left( \frac{1.6 \cdot E_{s}}{\sqrt{\frac{L_{v}}{OD_{post}}} \left( \frac{OD_{post}}{t_{post}} \right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_{s}}{\left( \frac{OD_{post}}{t_{post}} \right)^{\frac{3}{2}}} \right)$	F <sub>cr</sub> = 29 <b>ksi</b> AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3
Factored nominal shear resistance:	$\phi V_n \coloneqq \phi_{\mathcal{V}} \cdot 0.5 \ F_{cr} \cdot A_g$	$\phi V_n = 13.295 \text{ kip}$ AASHTO Eqn. 6.12.1.2.3c-1
Post Shear Check:	$\frac{\phi V_n}{V_{post\ u}} = 37.985 \qquad Post\_Shear\_0$	$Check := \text{if } \frac{\phi V_n}{V_{post\_u}} \ge 1.0$
	V post_u	"Post shear strength is satisfactory."
		"Post is not satisfactory."
	Post_Shear_	Check = "Post shear strength is satisfactory."
Following AASHTO 6.12.2.2.3 for Flexure De		
Check of Noncompact Section: Check_0	$Compact := \text{if } \frac{OD_{post}}{t_{post}} \le \frac{0.07 \cdot E_s}{F_y}$ $ \text{"Section is compact. Local buckling.}$	Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.
	else  "Section is not compact. Check wall	
Check_(	Compact = "Section is compact, Local buckling does	s not apply."



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Factored Nominal Moment Resistance:

$$\phi M_n := \phi_f \cdot F_v \cdot Z_{nost}$$

 $\phi M_n = 32.797 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{post\_u}} = 1.42$$

$$\frac{\phi M_n}{M_{post\_u}} = 1.42 \qquad Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \ge 1.0$$

$$\parallel \text{"Post flexural strength is satisfactory."}$$
else
$$\parallel \text{"Post is not satisfactory."}$$

Post\_Flex\_Check = "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area:

$$A_{\varphi} := A_{rail}$$

$$A_g = 0.54 \text{ in}^2$$

Distance from Max to 0 Shear:

$$L_v := \frac{L_{spc}}{2}$$

$$L_v = 48$$
 in

Critical Strength for Shear:

$$F_{cr} \coloneqq \min \left( 0.58 \cdot F_y, \max \left( \frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}} \left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left( \frac{OD_{rail}}{t_{rail}} \right)^{\frac{3}{2}}} \right) \right)$$

F<sub>cr</sub> = 29 **ksi** AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3

Factored Nominal Shear Resistance:

$$\phi V_n := \phi_v \cdot 0.5 \ F_{cr} \cdot A_g$$

$$\phi V_{n} = 7.832 \ kip$$

 $\phi V_n = 7.832 \text{ kip}$  AASHTO Eqn. 6.12.1.2.3c-1

Rail Shear Check:

$$\frac{\phi V_n}{V_{rail\_u}} = 41.052$$

$$\frac{\phi V_n}{V_{rail\_u}} = 41.052$$

$$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0$$

$$\text{"Rail shear strength is satisfactory."}$$
else
$$\text{"Rail is not satisfactory."}$$

Rail Shear Check = "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

Check Compact = "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance:

$$\phi M_n := \phi_f \bullet F_y \bullet Z_{rail}$$

 $\phi M_n = 13.339 \text{ kip} \cdot \text{in}$  AASHTO Eqn. 6.12.2.2.3-1

Post Flexural Check:

$$\frac{\phi M_n}{M_{rail\_u}} = 1.519$$

$$\begin{aligned} \textit{Rail\_Flex\_Check} \coloneqq & \text{if} \quad \frac{\phi M_n}{M_{rail\_u}} \geq 1.0 \\ & \quad \quad \| \text{``Rail flexural strength is satisfactory.''} \end{aligned}$$

Rail Flex Check = "Rail flexural strength is satisfactory."



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#### Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post:

$$w_{post\_wind} := f_{wind} \cdot L_{spc}$$

$$w_{post\_wind} = 120$$
 **plf**

Design moment from wind on post:

$$M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$$

$$M_{post\_wind\_u} = 21780 \ lbf \cdot in$$

$$M_{post\_u} = 23100 \ lbf \cdot in <- LL controls$$

Design shear from wind on post:

$$V_{post \ wind \ u} := \gamma_{WS} \cdot w_{post \ wind} \cdot H_{post}$$

$$V_{post\ wind\ u} = 0.66\ kip$$

$$V_{post\ u} = 0.35\ \textit{kip}$$
 <- LL controls

Uniform wind on rail:

$$w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$$

$$w_{rail\ wind} = 41.25\ plf$$

Design moment from wind on rail:

$$M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$$

$$M_{rail\ wind\ u} = 3960\ lbf \cdot in$$

$$M_{rail\_u} = 8778.966$$
 **lbf·in** <- LL controls

Design shear from wind on rail:

$$V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$$

$$V_{rail\_wind\_u} = 0.165 \text{ kip}$$

$$V_{rail\ u} = 0.191 \ \textit{kip}$$
 <- LL controls



On: 9/17/2021 Chk: RFA On: 12/22/2021

## Base Plate Design - Line Post w/ Axial Compression

Given:

Plans

Cap width:  $W_{cap} := 15.63 \ in$ 

Distance from post to end of cap:

Plate thickness:  $t_n := .5 \ in$ 

Plate length (perpendicular to fence):

Plate width (parallel to fence):

Compressive Strength of Concrete: Side clearance to anchor bolts:

Base plate steel yield strength:

Number of rails:

 $L_{end} := 8$  in

 $N_{plate} := 8 in$ 

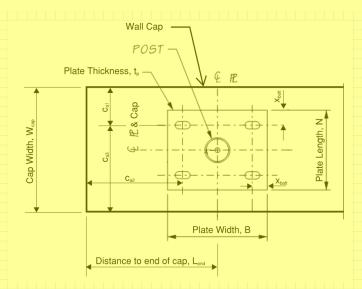
 $B_{plate} := 10 in$ 

 $f_c := 4 \, ksi$ 

 $x_{bolt} := 1.5 \ in$ 

 $F_{v plate} := 36$  ksi

 $n_{rail} := 4$ 



Output:

Plate Area:

Distance from bolt to near face of cap:

Distance from outside bolt to end of cap:

Distance from bolt to far face of cap:

Bearing Area taken to Be Same as Plate Area:

Max allowed bearing pressure line:

Max allowed bearing pressure:

Post dead load on plate

Rail dead load on plat:

Factored vertical load on plate:

Minimum length of area of bearing:

Critical eccentricity distance:

Eccentricity of loading:

 $A_{plate} := N_{plate} \cdot B_{plate}$ 

 $c_{al} := \frac{1}{2} \left( W_{cap} - N_{plate} \right) + x_{bolt}$ 

 $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ 

 $c_{a3} := W_{cap} - c_{a1}$ 

 $A_{bearing} := A_{plate}$ 

 $f_{pu\_max} := \phi_{brg} \cdot min \left[ 0.85 \cdot f_c \cdot \sqrt{\frac{A_{bearing}}{A_{closes}}} \right], 1.7 \cdot f_c$ 

 $q_{max} := f_{pu\_max} \cdot B_{plate}$ 

 $P_{post\ DL} := w_{post} \cdot H_{post}$ 

 $P_{rail\ DL} := n_{rail} \cdot 2 \cdot V_{rail\ DL}$ 

 $P_u := \gamma_{DL} \cdot (P_{post\ DL} + P_{rail\ DL})$ 

 $Y_{min} := \frac{P_u}{q}$ 

 $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ 

 $e_{loading} := \frac{M_{post\_u}}{P}$ 

 $A_{plate} = 80 \text{ in}^2$ 

 $c_{al} = 5.315$  in

 $c_{a2} = 4.5$  in

 $c_{a3} = 10.315$  in

 $A_{bearing} = 80 \text{ in}^2$ 

Conservatively setting bearing area to the same as the plate.

 $f_{pu max} = 2.04 \, ksi$ 

ACI Tbl. 14.5.6.1

 $q_{max} = (2.448 \cdot 10^5) \frac{lbf}{ft}$ 

 $P_{post\ DL} = 0.017$  kip

 $P_{rail\ DL} = 0.101 \ kip$ 

 $P_{y} = 0.148 \ kip$ 

 $Y_{min} = 0.007$  in

AISC DG#1 Eqn. 3.3.3

 $e_{crit} = 3.996$  in

AISC DG#1 Eqn. 3.3.7

 $e_{loading} = 156.322 in$ 

AISC DG#1 Eqn. 3.3.6

Small Moment Check := if  $e_{loading} \le e_{crit}$ Small moment check:

"Moment is small, no need for anchor bolts."

"Moment is large, need anchor bolts."

Small Moment Check = "Moment is large, need anchor bolts."

Distance from bolt to center of post:

$$f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$$
  $f_{dim} = 2.5$  in

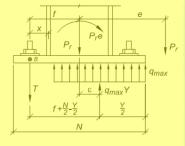


Fig. 3.4.1. Base plate with large moment.



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Plate dimension check

Plate Dim Check = "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} \coloneqq \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}} \qquad Y_{loading} = 0.179 \text{ in }$$

$$AISC DG#1 Eqn. 3.4.3$$

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$$T_u = 3.513 \ kip$$

 $T_u = 3.513 \text{ kip}$  AISC DG#1 Eqn. 3.4.2

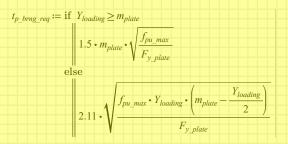
Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} \coloneqq \frac{N_{plate} - 0.8 \ OD_{post}}{2}$$

$$m_{plate} = 3.05$$
 in

Calculating minimum thickness based on bearing:



 $t_{p\_brng\_reg} = 0.366$  in

AISC DG#1 Egns. 3.3.14a-2 &

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$$x_{ten} = 1.55 \ in$$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$$p_{ten\_req} = 0.26$$
 in AISC DG#1 Eqn. 3.4.7a

Controlling minimum required base plate thickness:  $t_{p \text{ req}} := \max \left( t_{p \text{ bring req}}, t_{p \text{ ten req}} \right)$ 

$$t_{p\_req} := \max \left( t_{p\_brng\_req}, t_{p\_ten\_req} \right)$$

$$t_{p \ req} = 0.366 \ in$$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \ge t_{p\_req}$$
 \( \big| "Chosen plate thickness is adequate." \)
$$\text{else}$$
 \( \big| "Need a thicker plate."

Plate Thick Check = "Chosen plate thickness is adequate."



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#### Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size:

Min fillet weld size based on AISC Table J2-4

Chosen fillet weld size

Weld material:

 $F_{EXX} := 70 \text{ ksi}$ 

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter:

Effective Length of Weld:

Effective Throat Thickness:

Area of Weld:

Moment of Inertia of circular fillet weld:

Polar moment of Inertia of circular fillet weld:

 $L_g = 7.461$  in  $L_g := \pi \cdot OD_{post}$  $L_w = 6.836 \ in$ 

 $t_e := min\left(w \cdot \sin\left(45 \ deg\right), \frac{L_w}{4}\right)$   $t_e = 0.221 \ in$ 

 $A_w := L_w \cdot t_e$  $A_w = 1.511 \text{ in}^2$ 

 $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad I_w = 1.162 \text{ in}^4$ 

 $J_w := 2 \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e \qquad J_w = 2.325 \text{ in}^4$ 

AISC, Sect. J2, Pts. 2a & 2b

AISC, Sect. J2, Pts. 2a

17. Very thin annulus

 $A = 2\pi Rt$  $I_{\nu} = I_{\nu} = \pi R^3 t$  $r_{y} = r_{y} = 0.707R$  $J = 2\pi r^3 t$ 

Determine design strength of weld:

Nominal strength of weld metal:

 $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ 

 $F_w = 31.5 \ ksi$ 

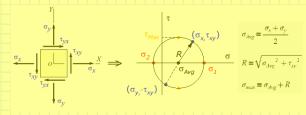
AISC, Tbl. J2.5

Normal stress caused by bending moment:

Stress caused by shearing force:

Resultant stress in weld from loading:

 $\sigma_b = 23.597 \text{ ksi}$   $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$ 



Check of weld thickness:

Weld\_Design\_Check := if  $F_w \ge \sigma_{max}$ 

"Chosen weld size is sufficient."

"Need bigger fillet weld."

Weld Design Check = "Chosen weld size is sufficient."

s\michael.fraker\documents\pw\_wsdot\dms26075\Pipe Rail Safety Fence Design Check - Cap Step Post.mcdx



Chk: RFA

On: 9/17/2021 On: 12/22/2021

**Anchor Bolt Connection Design** 

Given:

Number of anchor bolts resisting loads:

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter:

Bolt area:

Bolt nominal yield stress strength:

Bolt nominal ultimate tensile stress strength:

Bolt embedment:

Output:

Tension anchor bolt spacing:

Bolt nominal tensile stress strength:

Bolt nominal shear stress strength:

Ultimate tension load on one anchor bolt:

Required shear stress on one bolt:

Bolt modified nominal tensile stress strength, modified for effects of shear stress:

Bolt factored tensile resistance:

Only one side's bolts resist tension or shear.

 $d_{ab} \coloneqq \frac{5}{8} in$ Plans

 $A_b := 0.307 \ in^2$ 

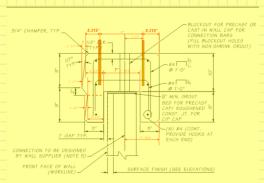
AISC Tbl. 7-18

 $F_{v \ holt} := 36 \ ksi$ 

AISC Tbl. 2-3

 $F_{u \ bolt} := 58 \ ksi$ 

 $h_{ef} := 5$  in



WALL CAP TYPICAL DETAIL

 $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{-t} - 1}$ 

 $F_{nt} := 0.75 \cdot F_{u \ bolt}$ 

AISC Tbl. J3.2

 $F_{nv} := 0.40 \cdot F_{u \ bolt}$ 

 $F_{nv} = 23.2 \ ksi$ 

AISC Tbl. J3.2, assuming threads within shear plane

Bolt\_Tensile\_Check = "Bolt is satisfactory."

 $T_{u\_ab} := \frac{T_u}{n_{ab}}$ 

 $f_{\rm v} = 0.57 \ ksi$ 

 $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_{v}\right)$ 

 $F_{nt}' = 43.5 \text{ ksi}$  AISC Eqn. J3-3a

 $\phi R_{n \ bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ 

 $\phi R_{n \ bolt} = 10.016 \ kip$ 

ACI 17.2.1.1

Check of bolt tensile stress:

 $Bolt\_Tensile\_Check := if \phi R_{n \ bolt} \ge T_{u \ ab}$ 

"Bolt is satisfactory."

"Bolt is no good."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor:

 $d_a := d_{ab}$   $d_a = 0.625$  in

Critical edge distance for adhesive anchors:

 $c_{ac} := 2 \ h_{ef}$   $c_{ac} = 10 \ in$ 

ACI 17.7.6

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: Group\_Tension\_Breakout\_Check := if  $s_1 \le 3 \cdot h_{ef}$ 

"Bolts act in group."

"Bolts act singly."

Group\_Tension\_Breakout\_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

 $A_{Nco} := 9 \cdot h_{ef}^{2}$   $A_{Nco} = 225 \text{ in}^{2}$ 

ACI Eqn. 17.4.2.1c

Actual projected influence area for bolt(s):  $A_{Nc} := min\left(\left(c_{al} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_{1}, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$   $A_{Nc} = 243.485 \ in^{-2} A_{Cl} \ Fig. R17.4.2.1$ 

Concrete tension breakout strength coefficient:

Value of 17 for post-installed anchors, per ACI 17.4.2.2



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Basic concrete tension breakout strength for single anchor:

$$N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf \qquad N_b = 12.021 \ kip$$

Factor for eccentrically loaded anchor bolts:

$$\Psi_{\alpha\alpha} := 1.$$

 $\Psi_{ec\ N} := 1.0$  Anchor bolts are not loaded eccentrically.

ACI 17.4.2.4

Factor for anchor bolts near an edge:

$$\Psi_{ed_{N}} := min \left( 1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{1.5 \cdot h_{ef}} \right)$$

$$\varPsi_{ed\_N} = 0.913$$

ACI Egn. 17.4.2.5b

Factor for anchor bolts in un-cracked concrete:

$$\Psi_{c N} := 1.4$$

 $\Psi_{c N} := 1.4$  Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.4.2.6

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_N} := min\left(1.0 \text{ , } \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right) \qquad \Psi_{cp\_N} = 0.75 \qquad \qquad \text{ACI Eqn. 17.4.2.7b}$$

$$\Psi_{cp\_N} = 0.75$$

Nominal concrete tension breakout strength:

$$\phi N_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b \qquad \phi N_{cbg} = 8.102 \ \textit{kip} \qquad \textit{ACI Eqn. 17.4.2.1b}$$

Check of concrete tension breakout failure:

$$\begin{aligned} \textit{Concrete\_Tension\_Breakout\_Check} \coloneqq & \text{if } \phi N_{cbg} \geq n_{ab} \cdot T_{u\_ab} \\ & \text{"Bolt is satisfactory."} \\ & \text{else} \\ & \text{"Bolt is no good."} \end{aligned}$$

Concrete\_Tension\_Breakout\_Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f_c}{2500 \ psi}\right)^{0.1} \cdot 2220 \ psi = 1512.441 \ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500 \ psi}\right)^{0.15} \cdot 2210 \ psi = 1541.429 \ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength:

$$\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$$

$$\tau_{uncr} = 1512.441 \ psi$$

Distance to edge of project influence area:

$$c_{Na} \coloneqq 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100 \ psi}}$$

$$c_{Na} = 7.329$$
 in

Check if anchor bolts act in group for bond failure:

Group\_Bond\_Failure\_Check := if 
$$s_1 \le 2 c_{Na}$$
 

"Bolts act in group.

Group\_Bond\_Failure\_Check = "Bolts act in group."



Chk: RFA

On: 9/17/2021 On: 12/22/2021

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Nao} := \left(2 \ c_{Na}\right)^2$$

 $A_{Nao} = 214.835 \text{ in}^2$ 

ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s):

$$A_{Na} := min\left(\left(c_{Na} + min\left(s_{1}, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{al} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$$

 $A_{Na} = 238.062 \text{ in}^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor:

$$N_{ba} := \tau_{uncr} \cdot \boldsymbol{\pi} \cdot d_a \cdot h_{ef}$$

 $N_{ba} = 14.848 \ kip$ 

ACI Eqn. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec\ Na} := 1.0$$

Anchor bolts are not loaded eccentrically.

ACI 17.4.5.3

Factor for anchor bolts near an edge:

$$\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{al}}{c_{Na}}\right) \qquad \qquad \Psi_{ed\_Na} = 0.918$$

$$\Psi_{ed\_Na} = 0.918$$

ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement:

$$\Psi_{cp\_Na} := min\left(1.0 \text{ , max}\left(\frac{c_{al}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$$

$$\Psi_{cp\_Na} = 0.733$$
Wall caps are not under l

ACI Eqn. 17.4.5.5b

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): 
$$\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$$
  $\phi N_{ag} = 7.192 \text{ kip}$ 

ACI Eqn. 17.4.5.1b

Check of bolt bond stress failure:

$$Bond\_Stress\_Check \coloneqq \text{if } \phi N_{ag} \ge n_{ab} \cdot T_{u\_ab}$$
 \(\begin{aligned} "Bolt is satisfactory." \\ else \\ "Bolt is no good." \end{aligned}

Bond\_Stress\_Check = "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout:

$$Group\_Shear\_Breakout\_Check \coloneqq \text{if } s_1 \leq 3 \cdot c_{a1}$$
 \(\begin{align\*} \text{"Bolts act in group."} \\ \text{else} \\ \text{"Bolts act singly."} \end{align\*}

Group Shear Breakout Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge:

$$A_{Vco} := 4.5 \cdot c_{al}^2$$

$$A_{Vco} = 127.122 \text{ in}^2$$

ACI Egn. 17.5.2.1c

Actual projected influence area for bolt(s): 
$$A_{Vc} := min\left(1.5 \cdot c_{al} \cdot \left(1.5 \cdot c_{al} + min\left(s_1, 3 \cdot c_{al}\right) + min\left(1.5 \cdot c_{al}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$$
  $A_{Vc} = 155.245 \ in^2$  ACI Fig. R17.5.2.1b

Load bearing length:

$$l_e := h_{ef}$$

$$l_e = 5$$
 in

ACI 17.5.2.2

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{al}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5 \ kip$$

ACI Eqns. 17.5.2.2a

Factor for eccentrically loaded anchor bolts:

$$\Psi_{ec}$$
  $_{V} := 1.0$ 

 $\Psi_{ec\ V} \coloneqq 1.0$  Anchor bolts are not loaded eccentrically.

Factor for anchor bolts near an edge:

$$\Psi_{ed_{\_}V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) \qquad \Psi_{ed_{\_}V} = 0.869$$

ACI Egns. 17.5.2.6a

Factor for anchor bolts in un-cracked concrete:

 $\Psi_{cV} := 1.4$ 

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

ACI 17.5.2.7



Chk: RFA

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Factor for small embedment

$$\varPsi_{h\_V} \coloneqq \min\left(1.0\;, \sqrt{\frac{1.5 \cdot c_{al}}{h_{ef}}}\right) \qquad \varPsi_{h\_V} = 1 \qquad \qquad \textit{ACI Eqn. 17.5.2.8}$$

Nominal concrete shear breakout strength:

$$\phi V_{cbg} \coloneqq \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b \qquad \phi V_{cbg} = 6.28 \text{ kip} \qquad \text{ACI Eqn. 17.5.2.1b}$$

Check of concrete shear breakout failure:

Concrete\_Shear\_Breakout\_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear:  $\phi N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right)$ 

$$N_{cpg} := min \left( \phi N_{ag}, \phi N_{cbg} \right) \qquad \phi N_{cpg} = 0$$

$$\phi N_{cpg} = 7.192 \text{ kip} \qquad A0$$

ACI 17.5.3.1

ACI 17.5.3.1

Concrete pryout strength in shear coefficient:

Nominal concrete pryout strength of anchor(s) in shear:

$$\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$$

$$\phi V_{cpg} = 14.384 \ kip$$

ACI Eqn. 17.5.3.1b

Check of concrete pryout strength in shear:

$$\begin{tabular}{lll} $Concrete\_Shear\_Pryout\_Check &:= & if & $\phi V_{cpg} \geq V_{post\_u}$ \\ & & & & & & & & \\ & & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & \\ & & & & & \\ & & & & & \\ & & & & & \\ & & & & & \\ & & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & \\ & & & \\ & & \\ & & \\ & & & \\$$

Concrete\_Shear\_Pryout\_Check = "Bolt is satisfactory."